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# TACHEOMETER SURVEYING

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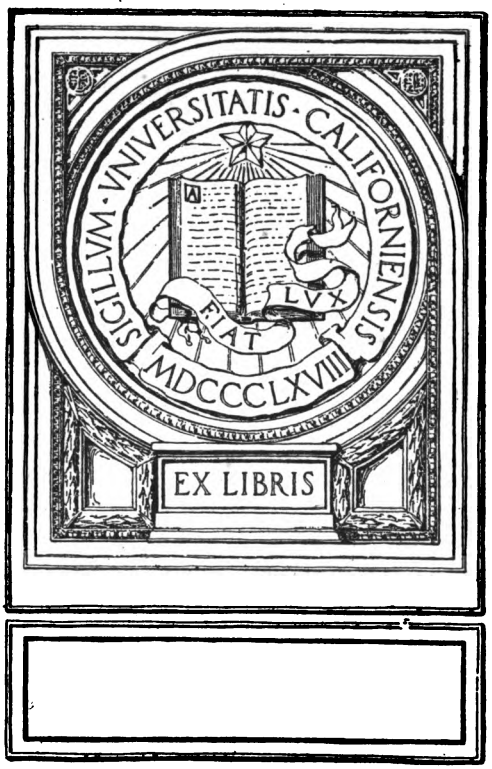


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# TACHEOMETER SURVEYING

## WITH SPECIAL NOTES ON PLOTTING, CARE AND ADJUSTMENT OF INSTRUMENTS, FIELD WORK, AND CALCULATIONS

BY

M. E. YORKE ELIOT,
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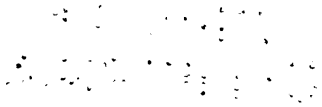

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# PREFACE.

THIS book is intended more especially for the information of those future members, or present junior members, of the engineering profession whose work has not yet familiarised them with the tacheometric methods now almost exclusively employed in making engineering surveys in countries other than England. Many treatises dealing comprehensively with the mathematics and theory of surveying have been written by high authorities upon the subject. These works —though of great value to surveyors, and to students who can combine the study of them with practice, under an experienced teacher, in field and office work—do not give such information as to the actual handling of instruments in the field, or the methods adopted for working out calculations in busy offices, as will enable the student to *teach himself*, the practice which is based upon theory. The author has endeavoured to supply this information in the following pages, and, to that end, has devoted four chapters to beginners in tacheometric work which contain preliminary explanations of the instrument, the methods of handling it, and other elementary matters. A contour survey made with the tacheometer is worked out—the operations, and the reasons for them, being explained step by step, from the selection and marking of the station points, through the field work, the booking of figures, and office calculations made for the production of the plan, to the calculation of lines and areas, etc., made from the data of the survey or from the plan itself.

Diversions are occasionally made to explain and illustrate



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alternative methods, but practically all the examples are taken from the complete field notes given.

The mathematics involved include nothing higher than the use of Chambers's mathematical tables as directed in the preface to those tables.

The penultimate chapter is devoted to the adjustments of the tacheometer, with a short description of recent and practical modifications of the instrument; in the final chapter the principles of construction of a slide-rule are elucidated, and the application of the instrument to the reduction of field notes is briefly explained, in the hope that students may be led to abandon the slow and laborious use of tables, at least for such purposes as this.

M. E. Y. E.

*November* 1915.

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# TACHEOMETER SURVEYING.

## CHAPTER I.

### *INSTRUMENTS.*

When a plan exists which represents upon a known scale some figure or enclosure, such as a field, the angles formed by the intersecting lines upon the plan can be measured with a protractor, or in some cases exactly calculated, and the lengths of the lines can be measured with a suitable scale. With these data, the area of the enclosure or figure can be ascertained, and the plan of it, if necessary, can be reproduced upon any other scale.

But when no plan exists, and consequently the ground itself which it is desired to represent by a plan must be measured, then instruments adapted to the direct determination of angles, distances, and heights, in the field must be used.

### Necessity for Special Instruments.

If the mere representation of boundaries and determination of areas are required, linear measurement with a chain or steel tape will suffice. Should the relative heights of only a comparatively few points be required in addition, the surveyor's level, in conjunction with the chain, is suitable. If it be desired to know for purposes of calculation, etc., the number of degrees, minutes, and seconds contained in each or any angle, the theodolite can be used in conjunction with



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the chain and level. But for engineering purposes a plan is usually required, not merely recording the shape and size of the piece of ground surveyed, but also representing all the undulations and variations of its surface. Certainly such a plan can be, and frequently is, made by means of the theodolite, level, and chain, or even level and chain alone. But the process is slow and involves much labour owing to the large number of points which must be determined (*a*) as to height with the level, (*b*) as to distance with the chain, and (*c*) as to direction with the theodolite, or by a second linear measurement.

These processes make the cost of such surveys excessive, and therefore a special instrument has been devised to enable the direction, distance, and altitude of all points within its range to be ascertained from one set of observations taken with it.

## The Tacheometer.

This instrument is known as a "Tacheometer." It consists of a transit theodolite, *i.e.* a theodolite fitted with a graduated vertical circle, and having a telescope of higher power than usual which can be completely revolved upon its horizontal axis; this telescope contains an optical device for taking linear measurements.

A delicate level-bubble is usually attached to the telescope itself. If there be, in addition to the ordinary lenses, a special one so arranged in the telescope as to refer all linear measurements taken with it to the *centre* of the instrument, this is called an "Anallatic" lens, or the telescope is said to be "Anallatic."

## Optical Measurement of Distances.

It is not proposed here mathematically to describe the optical principles involved in the measuring device, but a good idea of its working may be gained from the following :—

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If a piece of glass having two parallel lines ruled upon it is held at arm's length, and a graduated rod is viewed through this glass, the two ruled lines will appear to intersect the rod at points on its length whose distance apart will vary with the distance of the rod from the observer.

Thus, if twenty graduations are intercepted between the lines when the rod is 10 feet away from the eye, forty graduations will be similarly intersected when it is 20 feet away, the number of graduations subtended being doubled, because the distance is doubled.

The number of graduations, *i.e.* the length, seen to be intercepted between the lines ruled on the glass, thus gives a measure of the distance from the eye of the observer to the rod.

## The Stadia "Coefficient."

But a telescope is required in order to read such graduations at any considerable distance.

Accordingly, in the anallatic telescope of the tacheometer, lines, called "Stadia lines," either ruled on glass or formed of spider web, are arranged at such an interval apart that when a graduated rod is viewed through the telescope the distance from the centre of the instrument to the rod itself is a convenient multiple of the length on the rod seen to be intercepted between the stadia lines, and is usually a hundred times or two hundred times that length. This multiplier 100, or 200, etc., is called the "Stadia Coefficient."

## The Constant of the Instrument.

In the case of a telescope which has no anallatic lens, distances thus obtained are not from the centre of the instrument to the rod, but from a point in front of the object-glass to the rod. The distance of this point from the object-glass is equal to the focal length of the latter.

To obtain the full measurement from the centre of the

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instrument to the rod, an addition must therefore be made to each distance measured with a non-anallatic telescope. The figure to be added is called the "Constant" of the instrument; it is the sum of the focal length of the object-glass and the distance of the object-glass from the centre of the instrument. If it be not given by the maker, its value can be found as follows:—Focus the telescope on a very distant object. Measure from the centre of the O.G. to the screws holding the diaphragm, and from the centre of the O.G. to the centre of the instrument; the sum of these two measurements is the constant of the instrument.

## Calculation of Distances from Stadia Readings.

It is evident from what has been said that if, on looking through an anallatic tacheometer telescope, placed horizontally, and having stadia lines set 1 : 100, it is seen that one stadia web cuts a graduated rod, divided into feet and hundredths, at the 1′·00 mark, while the other cuts it at the 2′·00 mark, then the distance of the rod from the centre of the instrument is the difference between these two readings, viz. 1′·00, multiplied by 100 = 100 feet.

Were the wires set for 1 : 200, they would have subtended only 0·50′ on the rod, which, multiplied by 200, again gives 100′ for the distance to the rod.

And if the telescope has no anallatic lens, the wires would subtend slightly less, and the addition of the "Constant" would be required.

If the constant were 1 foot, then at 100 feet distance the 1 : 100 wires would subtend 0·99 feet, and $(0{\cdot}99 \times 100 + 1{\cdot}00) = 100$ feet as before.

The advantage of the anallatic lens is that it saves the addition of the constant; the disadvantage is that the image is not quite so bright as with a non-anallatic telescope. Opinion is divided on the point, but the writer prefers the

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anallatic type. Good work can certainly be done with an ordinary transit theodolite having stadia lines marked on its diaphragm, but the telescope of such an instrument is usually of low power, and this unduly restricts the working range.

## Working Range of Tacheometers.

Experience has shown that, owing to difficulties connected with the manufacture of lenses, etc., the limit for accurate determinations of distance with an ordinary 5-inch tacheometer is about 600–800 feet.

For approximate purposes, and with staves or rods having bold graduations, this limit may nevertheless be notably exceeded. In fact, given clear weather, and such an adaptation of the stadia principle to a very powerful telescope as that provided in the "Magnum" tacheometer, designed by Mr T. Graham Gribble, M.Inst.C.E., it may even be trebled. (See Chapter VIII.) Great accuracy, combined with long range, can be obtained by the use of a micrometer eyepiece, observations being taken on to a fixed base indicated by targets attached to a rod. But owing to additional complications in working out results, and a certain amount of delicacy in construction, the micrometer is not so generally useful a distance-measuring device as the ordinary stadia diaphragm. The latter has the advantage of simplicity, combined with sufficient accuracy for most observations; hence it is the best all-round tool.

Up to about 700 feet, and with a good telescope of 13-inch focal length and 1·8-inch aperture, distances may be read by means of stadia wires within $\frac{1}{1000}$ of the truth, thus comparing favourably with all but the *best* chainage.

Fig. 1 shows common arrangements of the webs in a tacheometer telescope, while fig. 2 shows the appearance of an ordinary "Sopwith" levelling staff, as viewed through it.

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The reading given by the 1 : 100 stadia wires in this figure is :—

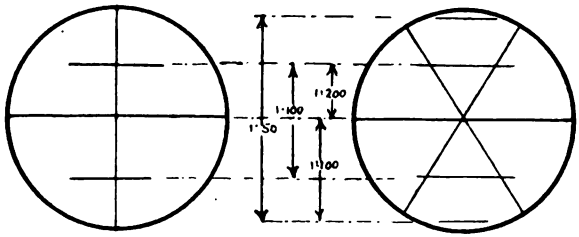


FIG. 1.—Tacheometer Diaphragms.

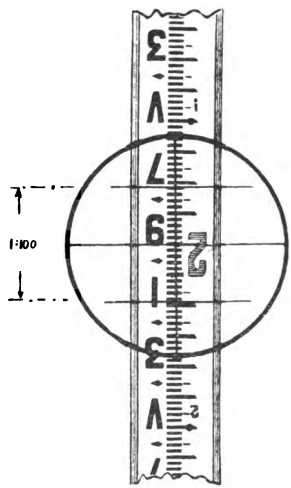


FIG. 2.—Levelling Staff, as viewed through Telescope.

| | |
|---|---|
| Upper stadia web . . . . | 1·71 |
| (Central web 1·90) | |
| Lower stadia web . . . . | 2·09 |
| Difference . . . . | 0·38 |

which multiplied by 100 gives 38 feet.

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This distance could have been obtained from the central web reading and that of either stadia web, by using twice the value coefficient, thus :—

| | |
|---|---|
| Upper web . . . | 1·71 |
| Central web . . . | 1·90 |
| Difference . . | 0·19 × 200 = 38 feet. |

The central web reading is the arithmetical mean of the two outer web readings when the line of sight is horizontal or approximately so. (See Chapter IV.)

The image seen in the telescope is upside down, owing to the use of an inverting eyepiece. Erecting eyepieces are seldom used in practice (at all events by British engineers), owing to the loss of light caused by the extra lenses, and to the inconvenience arising from the additional length. Their cost is also excessive.

The observer very soon becomes accustomed to the ordinary inverting eyepiece, so that he no longer notices the inversion of the image in the telescope.

## Construction and Use of Verniers.

The line of sight denoted by the intersection of the cross webs, or cross lines ruled on glass, in the telescope of a theodolite can be directed to any desired point with great precision, and, therefore, in order that angles may be read upon the circles of the instrument with a corresponding degree of accuracy, *vernier scales* are fitted to the graduated circles.

In examining the construction and use of these scales, it will be well, for the sake of simplicity, first to consider a vernier made to read an ordinary straight scale.

## Vernier for Straight Scale.

Fig. 3 shows part of a scale AB divided into centimetres only. Upon it, except by estimation, one centimetre is the

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smallest length that can be measured. But above AB is a vernier scale CD upon which a length of 9 centimetres is divided into ten spaces instead of nine. This scale CD is supposed to be capable of sliding along AB.

Now, since on the scale CD there are ten divisions in 9 centimetres it follows that when the zeros of the two scales coincide, the first division on CD is exactly $\frac{1}{10}$ or 0·1 centimetre behind the first one upon AB, its second division mark is $\frac{2}{10}$ or 0·2 centimetre behind the nearest one to it on AB, and the third, fourth, fifth, etc., division marks on CD are respectively 0·3 centimetre, 0·4 centimetre, 0·5 centimetre, etc., behind the graduations on AB which are nearest to them. Keeping AB stationary, and moving the vernier scale CD to the left, it will be seen that when the zero point of the latter is, say, 0·4 centimetre beyond that of the former, then the vernier division marked 4 will exactly coincide with a division on the lower scale, and similarly when the vernier zero is at 0·9 centimetre beyond the scale zero, the vernier division marked 9 will coincide with a division on the scale. Hence, in order to ascertain the exact number of vernier units, which in this case are tenths of a centimetre, by which the vernier zero has passed beyond any whole division of the lower scale, it is necessary only to see which of the vernier divisions is exactly coincident with a scale division, and the number of $\frac{1}{10}$ths of a centimetre sought will be indicated by the value marked against that vernier division. With such a device, the scale being graduated to single centimetres only, $\frac{1}{10}$th of a centimetre can be read with exactitude.

## Theodolite Verniers.

In verniers which read into scales of degrees, etc., on the circles of a tacheometer or theodolite, the foregoing principle is applied, in order to give readings to single minutes of arc or less. Instead of a mere arrow-point, indicating only whole divisions of the graduation, each circle is fitted

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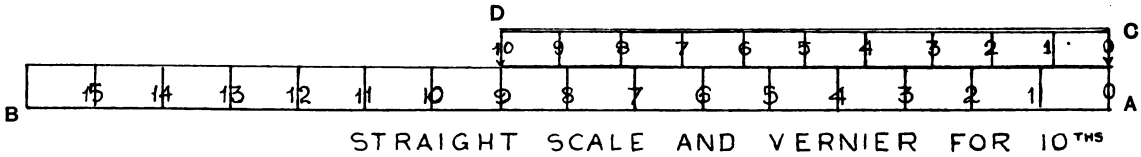


FIG. 3.—Straight Vernier.

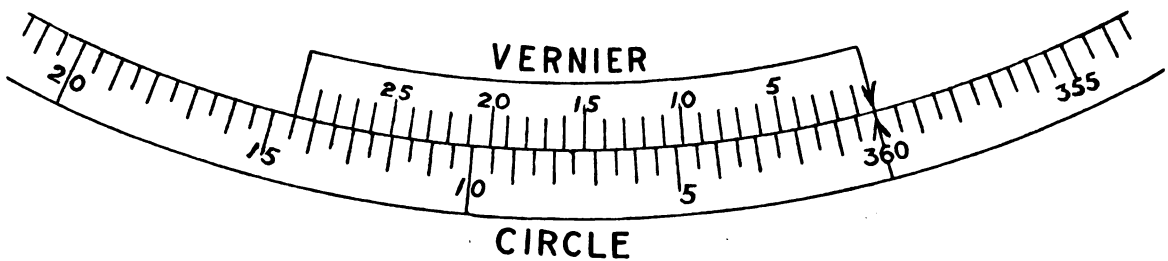


FIG. 4.—Theodolite Vernier.

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with one or more small vernier scales, with arrow-marks placed at their zeros, as in the diagram fig. 4 (in which the width of the divisions is exaggerated for the sake of clearness).

The smallest subdivision on the (horizontal) circle (shown in the figure) is $\frac{1}{2}$ degree, or 30 minutes, but the vernier V, which revolves with the upper part of the instrument, makes it possible to read single minutes of arc as follows :—

The scale on the vernier extends over twenty-nine open divisions ($\frac{1}{2}$ degrees) of the circle, or $14\frac{1}{2}$ degrees, but in this length it has thirty divisions. When the zeros of the vernier and the circle are brought to coincide, the first division on the vernier will be $\frac{1}{30}$ of a circle division (or $\frac{1}{30}$ of $30' =$ 1 minute) behind the $\frac{1}{2}$-degree mark next in advance on the circle scale below it, while the second, third, fourth, etc., vernier divisions will stand $\frac{2}{30}$, $\frac{3}{30}$, $\frac{4}{30}$ of a $\frac{1}{2}$ degree, or $2'$, $3'$, $4'$, and so on, behind the $\frac{1}{2}$-degree marks next in advance of them on the circle. If, therefore, the vernier zero is moved so that it stands successively at points 1, 2, 3, etc., minutes beyond an exact half-degree mark, then the vernier divisions for 1, 2, 3, etc., minutes will successively become coincident with a division on the circle scale below them. To read an angle to minutes with such a vernier, first note the number of whole degrees (and possibly an odd half-degree) on the circle, indicated by the arrow at the vernier zero mark, and then look along the vernier scale, to see which of its divisions coincides exactly with one on the circle scale below it; the value marked on the vernier against this division gives the number of odd minutes to be added to the whole degrees (and possibly a half-degree). A vernier made to read single minutes will usually be numbered at each 10 minutes of its length, in order to avoid crowded figures. The figure of the nearest graduation behind the coincident division is, therefore, first looked for, and the remaining minutes are counted up to the coincident

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division and added to this figure, the complete vernier reading being then added to the whole degrees (and possibly a half-degree) indicated on the circle scale, by the arrow at the vernier zero. The closest reading which can be taken with an ordinary vernier is :—

$$\frac{\text{(Value of smallest division on circle or scale)}}{\text{(Number of divisions on vernier)}}.$$

Conversely, in constructing a vernier to read to any desired unit :—

Number of unit divisions required on vernier

= Number of units contained in one division of the scale.

The length of the vernier will be equal to that of as many divisions of the scale, minus one, as there are to be divisions on the vernier.

Thus if each of the degrees marked on the (horizontal) scale contains two subdivisions (each of 30′), and there are sixty divisions on the vernier, it will read to :—

$$\frac{30'}{60'} = \tfrac{1}{2} \text{ minute, or } 30''.$$

Its length will be equal to $(60-1)$ half-degrees, or $29\frac{1}{2}$ degrees on the scale into which it reads. Another vernier reading to 30″ might cover 13 degrees on the circle, if the value of the smallest circle division were 20′. In this case there would be forty divisions on the vernier scale, covering thirty-nine divisions of 20′ each on the circle.

A vernier reading to 20″, the circle being divided to 20′, would cover fifty-nine divisions (19° 40′), and have sixty divisions. A magnifier is usually attached over each vernier to facilitate reading. For the measurement of angles in the vertical plane, a graduated circle is fitted to, and revolves with, the horizontal axis of the telescope. The verniers reading into this circle are necessarily fixed, as the circle itself revolves, but they are otherwise similar to those of the horizontal circle.

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## CHAPTER II.

### *THE HANDLING OF INSTRUMENTS*

BEFORE attempting the simplest kind of survey with his instrument, the absolute tyro must handle it for a while in order to familiarise himself with the functions of its various screws and fittings. (See fig. 5.)

### Adoption of Routine.

Every beginner is recommended, from the inception of his instrumental work, to adopt a routine method of handling his tools, and to adhere to it. This will tend to the avoidance of accidents and mistakes which may otherwise arise out of a moment's inattention. Many valuable instruments have been broken by picking them up without noticing that the screws or caps, which hold the telescope in the V supports, or the whole instrument to the stand, had been left unfastened. Probably as many more have been damaged by incorrect repacking in the box, with subsequent attempts to force the lid to close. When a new, or unfamiliar, instrument arrives, it is advisable to write in pencil the names of a few of the principal fittings against the position in the box where they lie, *before* taking anything out.

When handling the various screws, etc., no force must be used. A very light pressure suffices; unnecessary force will result in damage, or in moving the whole instrument on its stand, thus rendering accurate work impossible. The



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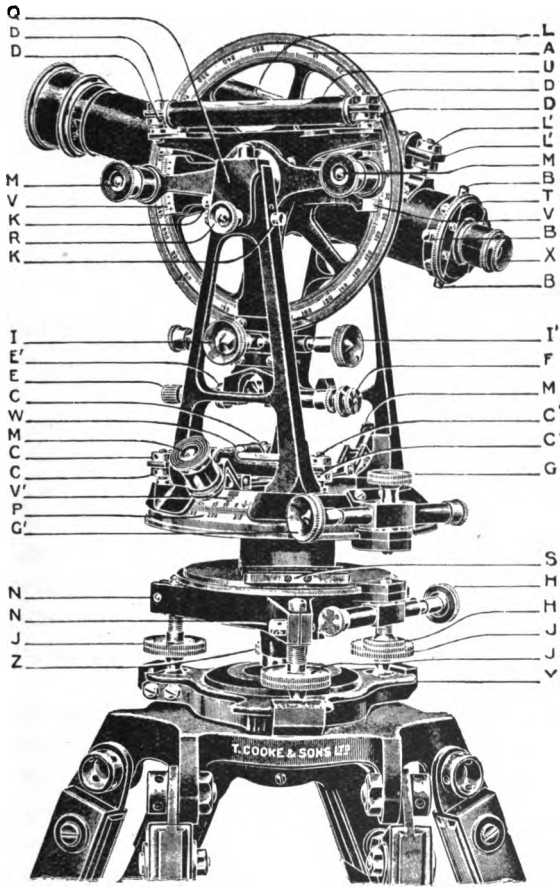


**FIG. 5.—General View of Tacheometer.**

A. Vertical circle. B, B, B. Adjusting screws to diaphragm. C, C, C. Capstan-headed nuts for adjusting main plate bubble. C′,C′. Capstan-headed nuts for adjusting small plate bubble. D, D, D, D. Capstan-headed nuts for adjusting azimuth bubble. E. Clipping screw for adjustment of verniers to vertical circle. E′. Locknut to clipping screw. F. Quick-release opposing spring to clipping screw. G. Clamping screw of upper limb. G′. Slow motion (tangent) screw to upper limb. H. Clamping screw of lower limb. H′. Slow motion (tangent) screw to lower limb. I. Clamping screw of vertical circle. I′. Slow motion (tangent) screw to vertical circle. J, J, J. Levelling screws (footscrews). K, K. Screws for vertical adjustment of horizontal axis. L. Level-bubble on telescope. L′, L′. Capstan-headed nuts for adjusting level-bubble. M, M, M, M. Magnifiers for reading verniers. N, N. Screws for taking up wear of footscrews in tribrach arms. P. Horizontal circle. Q, Q. Caps to trunnions. R. Securing screws of caps to trunnions. S. Trough compass attachment. T. Screw which moves eyepiece across diaphragm. U. Azimuth bubble. V, V. Verniers to vertical circle. V′. Vernier to horizontal circle. W. Main plate bubble. X. Screw focussing eyepiece. Y. Spring locking plate. Z. Locking plate clamping screw.

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fingers should be kept away from all graduated surfaces and lenses, nor should the instrument be touched with perspiring hands if it can be avoided.

### Setting up Tacheometer or Theodolite.

The tripod must be firmly planted on the ground and the points pushed into the soil until a firm base is obtained. The lower part of the instrument, after noting the position of its various parts, as packed, so as to facilitate replacement, is then taken from its case and placed upon the tripod stand, without the telescope, which is usually packed separately. In order to fasten it securely to the stand, the clamp screw Z (fig. 5) is slackened, and the locking plate Y pushed to one side with the thumb so as to permit the rounded ends of the footscrews J, J, J to slide into the recesses provided for them. Thereafter the plate Y is allowed to spring back into place, and the screw Z is tightened, leaving the instrument securely clamped to the tripod. At this stage the instrument can be very roughly levelled by moving the tripod legs while referring to the horizontal plate bubbles.

The caps Q, Q are now turned back, and the telescope with its attachments is taken from the case. The vertical arc clamp screw I is slackened, and the arm turned downwards approximately at right angles. The telescope is then carefully lowered into position in the V-shaped bearings, in such a way that the projection on the cross bar of one of the standards comes between the clipping screw E and its opposing spring F. A partial turn of F releases the spring which presses against the projecting stop, and holds the arm perfectly rigid. The caps Q, Q are returned over the ends of the axis and fastened there with the screws R, R; the trough compass is placed in position within the slide S, and the plumb-bob is hooked to the ring beneath the vertical axis of the instrument, which can now be safely moved from place to place. The vertical circle should, if possible, be in the

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position it occupied when the instrument was adjusted, as its weight and that of its fittings is not always properly counterbalanced except in the best instruments. If the circle stood over the vernier marked A when the level-bubbles were adjusted, it is best to work with it over the same vernier, as changing it to the other side may throw the bubbles slightly out of truth. (See Chapter VIII.)

The next step is to place the horizontal and vertical axes of the instrument exactly horizontal and vertical.

### The Level-Bubbles.

In the theodolite, which has no level-bubble on the telescope, this is done by reference to the bubbles (fig. 5) upon the horizontal plate. But in the tacheometer a more delicate level L is fixed to the telescope, and it is better to use this bubble for the purpose. The plate bubbles, owing to partial shade thrown by the A-shaped supports, differences of temperature, or want of exactitude in adjustment, may not be in perfect agreement with it. In the course of a day's work, especially in hot climates or on sunny days, some variation between the bubbles will probably be found ; it is therefore recommended that the large telescope bubble should invariably be the criterion in tacheometric work, or, in the event of this bubble being broken, then the bubble against the vertical arc should be used, except for making the preliminary and approximate adjustments when levelling up the instrument, for which the plate bubbles are sufficiently accurate.

### The Vertical Circle Vernier Zeros.

It must not be forgotten that, when putting the telescope axis into the trunnions, one of the clipscrews E, or rather the spring F which replaces one of them in modern instruments, was moved. On the exact adjustment of the clipping arm depends that of the zero lines on the vertical arc verniers,

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and although a spring clamp is used instead of an opposing screw, reliance cannot be placed upon its returning exactly to the correct position on every occasion.

The first time an instrument is levelled up after being taken from its box, it is therefore necessary to examine this adjustment in addition to levelling up the instrument. Thereafter it will not (or should not) require attention unless the telescope is removed from the trunnions to repack it in the box, or should the wrong screw be moved by mistake, while it can be disregarded altogether if no vertical angles are to be read.

The so-called permanent adjustment of the level-bubbles, diaphragm, and other parts of the instrument which may become deranged by rough handling or through other causes, should not be attempted by an inexperienced person. It is, of course, desirable that the surveyor should, as soon as possible and under proper supervision, learn to keep his instrument in perfect adjustment, without which he can have no certainty as to the accuracy of his work. A full description of the method is given in Chapter VIII.

## Levelling the Instrument.

Presuming the permanent adjustments to be correct, the process of levelling the instrument, after setting it up, is as follows :—

Release the clamp screw H of the lower limb and turn the instrument so that the longer level-bubble W on the horizontal plate lies in the position $a$–$a'$ (fig. 6), parallel with two of the footscrews of the instrument. Bring this bubble to the centre of its run, either by turning these two footscrews in opposite directions, one with each hand, if it is far out, or by turning one only if it is nearly in the middle. Next turn the same bubble so that it lies over the other footscrew, in the position $b$, $b'$, and with this screw only bring the bubble again to the centre of its run. Return to the

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position $a$, $a'$, and repeat these operations until the bubble remains in the centre of its run in both positions.

If the bubble W is in perfect adjustment, it will now remain at the centre of its run while the instrument is completely revolved upon its vertical axis.

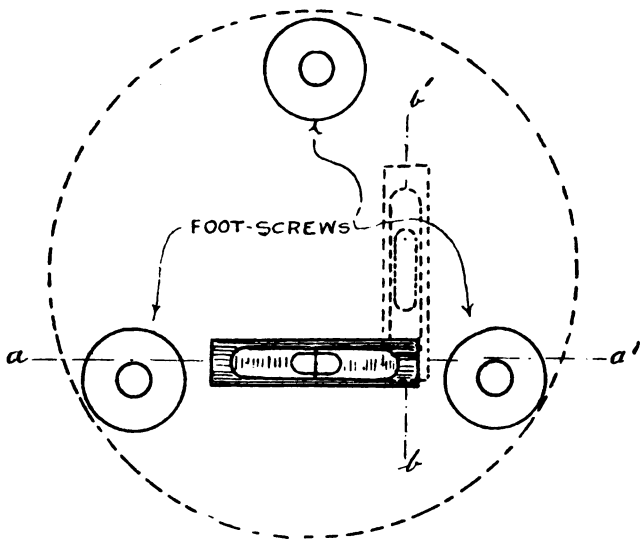


FIG. 6.—Positions of Bubbles and Footscrews when levelling up.

## Adjustment of Vernier Zeros.

If levels are to be taken with the tacheometer, the final levelling up and the adjustment of the vertical circle vernier zeros should be completed by reference to the very sensitive bubble on the telescope. Tighten the lower clamp H and loosen the upper one G. Bring the telescope bubble to the position $a$, $a'$ (fig. 6) over two footscrews. With the clamp screw I slackened, turn the telescope by hand so that the horizontal diameter of the vertical circle and the vernier zeros are approximately coincident, then tighten I, and make



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them exactly so by means of the tangent screw I′. If the vertical circle is figured from zero at the top, right round the circle to 360, its horizontal diameter will pass through the 90° and 270° divisions, or it may, as is sometimes the case, be figured in four quadrants reading 90° in each direction and having two zeros on the horizontal diameter.

Next release the locknut E′, and then, by means of the clip-screw E, bring the bubble on the telescope to the centre of its run. Note that upon the completion of this adjustment the locknut E′ must be gently tightened up again, to prevent any unintentional movement of the clipscrew E. Now turn the telescope bubble to the position *b*, *b′* and correct it with the third footscrew only. Return the bubble to *a*–*a′*, and, if it be no longer central, readjust with one only of the two footscrews over which it lies. Then turn the upper limb through one-half of a revolution, thus bringing the bubble back to *a*–*a′*, but reversed end for end.

If it leaves the centre of its run, correct half the error with one of the footscrews, and half by the clipscrew E. Again correct with the third footscrew only, and continue the process of reversing and correcting until the telescope bubble remains central in all positions throughout a complete revolution of the upper limb, when it will be found that the bubble fixed to the vernier arms will also remain in the centre of its run. Examine the verniers occasionally during the operation to make sure that they are still set at zero on the vertical arc, as the tangent screw I′ may have been touched in mistake for the clipscrew E.

The final levelling up is always made by means of the *clip-screw E* and the footscrews, with the vertical arc set to zero, when the instrument is taken from the box and set up for the first time in the day's work, its object being the adjust-ment of the vernier zeros. Thereafter, the clipscrew is clamped by the locknut and should not be touched, but the vertical circle should be set to zero and the instrument

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levelled up by the telescope bubble, with the footscrews only. The verniers will remain in the correct position until the telescope is again removed from the trunnions unless the clipscrew is accidently moved, in which case neither vernier bubble nor telescope bubble can be made to "reverse" correctly with the footscrews only. The vernier bubble acts as a check upon the position of the vernier zeros when the telescope is elevated or depressed.

The assumption has been made above that the permanent adjustments of the instrument are correct; but those of the collimation lines in particular are so important that they should be tested frequently (see p. 120) to ensure that the line of sight is truly horizontal when the telescope bubble is in the middle of its run with the vertical circle at zero, and that the line of sight through the vertical web is at right angles to the horizontal axis.

## Practice for Beginners.

Having set up the instrument, levelled it, and adjusted the vertical circle verniers correctly, it is ready for work.

It will be advantageous for the beginner to practise a little, setting up his instrument firmly so that the point of the plumb-bob is exactly over a mark, levelling it, and taking readings on the verniers at various points of both circles, and through the telescope on to a graduated staff fixed against a tree or wall, until he is able to unpack his instrument from the box, set it up ready for work in a few minutes, and take vernier and staff readings without difficulty.

## Vernier Readings.

Perhaps as good a method as any of practising vernier readings is the following:—

Clamp the lower plate, set the zero of one of its verniers as nearly to 360° as can be judged by eye, and then clamp

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the upper plate. Focus the magnifier M so that the graduations are seen very distinctly. Now, with the tangent screw G′, bring the zero line of the vernier exactly in line with the 360° mark.

Notice that the first graduation on the vernier is very slightly behind the line beneath it on the horizontal circle, and that each succeeding line is rather more behind that one preceding it, until the lines again become coincident at the last vernier division. See explanation of verniers, p. 7 *et seq.*

Now turn the tangent screw very slightly, and make the first vernier division beyond zero correspond with the one on the circle just in advance of it. The vernier, which is assumed to be graduated for half-minutes, now reads 0° 0′ 30″, or one vernier unit. Turn the screw till the second division coincides with the one on the circle which is just in advance of it. The reading is then two vernier units, or 0° 1′ 0″. Turn gently on, reading the vernier, half-minute by half-minute, throughout the first half-degree (0° 30′ 0″). Then slacken the clamp-screw G, turn the upper part of the instrument gently by hand so that the verniers are successively at various points on the circle, and read the angles indicated, not forgetting to add the whole degrees (and possible half-degree) passed over from 360° to the odd minutes and seconds read on the vernier.

## Staff Readings.

In practising with a graduated staff or stadia rod for the first time, examine the graduations carefully. On a "Sopwith" staff (fig. 7) notice that the tops of the figures marked in black upon the right-hand side indicate complete *odd* tenths (·1, ·3, etc.), and the bottoms of them complete *even* tenths (·2, ·4, etc.) of a foot; that the figures for *whole* feet are in red, and have their tops at the complete foot they indicate. Note where the *joints* of the staff are, as they

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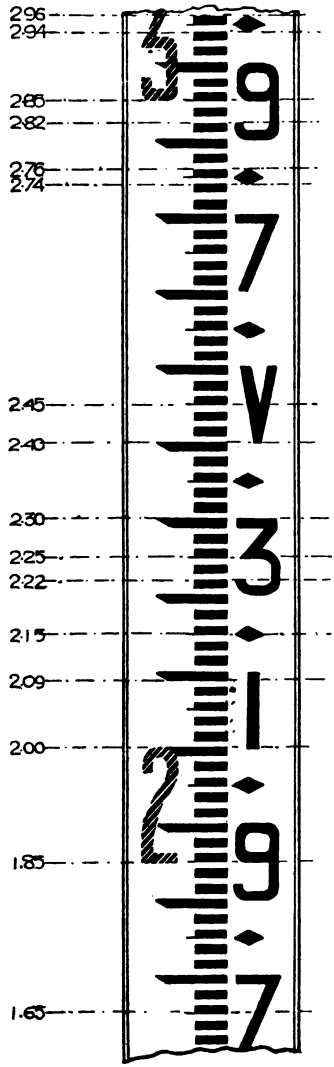


FIG. 7.—Sopwith Levelling Staff.

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will help—in distant readings when the red figures cannot be very clearly seen—to determine the whole foot which is to be booked.

Try moving a pencil-point up and down a staff, placed *upside down* (in which position its image will appear in the telescope), stopping the pencil at even tenths and reading the value indicated in feet and tenths. When this can be easily and quickly done, stop the pencil-point at intermediate points between tenths, and practise thus until the value indicated at any point can be read without hesitation. Then focus the telescope of the instrument on the staff at about 100 feet distance, so that the graduations are quite sharply and distinctly seen through it. Move the eyepiece slightly in or out of its socket until the cross hairs of the telescope are clearly defined as sharp black lines, which seem to be fixed across the staff, and do not appear to move when the eye is shifted slightly up or down in looking through the telescope. The apparent motion of the wires when the eyepiece is not properly focussed is called " parallax," and its removal is essential.

Now tighten the clamp-screw I, and, with the tangent screw I′ elevate or depress the telescope, taking a half-turn or so at a time, and then stopping to read the graduation of the staff where the central horizontal wire cuts it. Fig. 7 shows part of an ordinary levelling staff with various readings marked against it.

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## CHAPTER III.

### *THE SURVEY OF A SIMPLE ENCLOSURE, OR FIGURE BOUNDED BY STRAIGHT LINES.*

WHEN the student is able to level up his instrument over any given point, and to read the verniers and graduated staff correctly, he is fit to undertake a tacheometrical survey of an area enclosed by a few straight boundary lines, involving the determination of horizontal distances and angles only. An assistant is needed to hold the staff.

The kit required consists of :—

Tacheometer and staff.
Field book and pencil.
Flag-poles and wooden pegs.
Blue pencil for marking pegs.

### Form of Field Book.

Although, for first attempts, an ordinary note-book can be used for entering readings, it is better to have a properly ruled field book. None of those commonly sold for the purpose seem to be really satisfactory ; the form recommended is given below.

In any case the book should have stiff covers, be printed on good paper, and allow plenty of width in the columns for entering figures ; it should not be bulky, since it is frequently necessary to carry two or three such books in the pocket—one in use, and the others for reference to past work. The form here illustrated was drawn up by the



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(Outside cover.)

TACHEOMETER BOOK No.....

SURVEYOR.....

.....SURVEY

Date.....

(Left-hand inside page.)

NAME AND PLACE OF SURVEY.....

| Station. | | Bearing. | Stadia Wires. | Obsd. Dist. | Hor. Dist. | Vert. Circle. | Height of Inst. | Reduced Level of Inst. Collimation. |
|---|---|---|---|---|---|---|---|---|
| From | To | ° ′ ″ | | | | | | |
| | | | | | | | | |

(Right-hand inside page.)

BOOK No..... PAGE No.....

SURVEYOR..... Date.....

| Line of Sight. | | R.L. of Line of Sight at Staff. | Centre Wire Staff Reading. | Reduced Level of Point observed. | Remarks. |
|---|---|---|---|---|---|
| Rise. + | Fall. − | | | | |
| | | | | | |

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engineers of a Colonial Government Department. Each book contains about eighty leaves, the covers are of stiff cardboard, and are *not* dyed, as the colours commonly used are apt to " run " badly if they become damp.

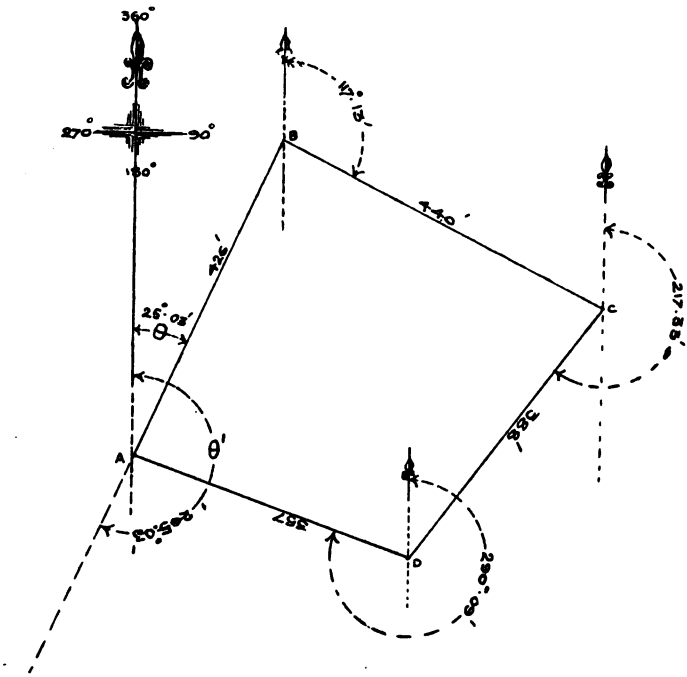


FIG. 8.—Simple Four-sided Enclosure.

The following method can be adopted for purposes of practice :—

In some open and fairly level space select, say, four points to represent the angles of the proposed straight-sided enclosure, and drive a peg firmly into the ground at each point selected.

With an ordinary blue chalk pencil mark these pegs consecutively A, B, C, and D (fig. 8), and thrust one of the flag-poles or rods firmly into the ground at B, C, and D.

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Then set up and level the tacheometer over the centre of the peg marked A.

Bring the verniers on the horizontal circle to zero, leave the plates clamped together, and release the lower clamp screw H. Free the compass-needle by means of the screw provided for that purpose, and revolve the whole instrument on the lower axis, until the compass-needle points very nearly to the centre of the graduations at the end of the box. Then tighten the lower clamp screw H, and, with the lower tangent screw, H′, make the North end of the compass-needle point exactly to the central division (or to 360 if a circular compass is fitted to the instrument).

The telescope is now sighted along a magnetic North and South line, or is "in the magnetic meridian." From this line angles will be measured.

Focus the eyepiece of the telescope so that all "parallax" is removed, and let the flag at B be held vertically with its point on the centre of the peg.

Release the upper clamp G, leaving the lower clamp fast, and, looking through the telescope, bring the intersection of the cross wires approximately in line with the flag at B, turning the upper part of the instrument by hand. Release the vertical circle clamp, I, and raise or depress the telescope so that the flag-pole is viewed as near its *lower* end as possible, because it may not be quite vertical.

Clamp the vertical circle and the upper plate, and with the tangent screw G bring the intersection of the cross webs to bear upon the centre of the flag-pole.

The vernier, which was originally set at 360°, now indicates the angle $\theta$ in fig. 8 which the line AB makes with the magnetic meridian passing through A.

### Bearings and Back Bearings.

This angle is known as the "bearing" from A to B, or, briefly, the "bearing AB."

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Bearings are measured clockwise round the circle, 90° being due East, 180° due South, 270° due West, and 0 or 360° due North. The other vernier indicates the angle $\theta'$. This is the back bearing of the line AB, *i.e.* the bearing B to A; it is simply the forward bearing AB ± 180°. Enter the bearing AB (25° 03′ 00″ in the figure) in the "Bearing" column of the field book as shown below, A and B being placed upon consecutive lines in the "Stations From and To" columns, in order that the system of making entries may be the same as that which is used when levels are also taken, in which case a separate line is required for each point. Next let the graduated staff be held vertically upon peg B and be read with the telescope horizontal.

The central horizontal line in the telescope intersects the staff at 3·33, one of the 1·100 stadia lines shows 1·20, and the other is at 5·46. The stadia readings are booked opposite B (since this is the station to which we are observing), as shown in the appropriate column. The centre wire reading is entered by itself in the column reserved for it; if the stadia lines have been correctly read, the mean of their readings will be the centre wire reading.

The line AB has now been determined both as to length and direction, and the instrument is removed to B, after clamping the telescope in a vertical position and releasing the clamp of the lower limb, for greater safety in transport. A flag is left just *behind* the peg A, so that a line drawn to it from B would pass over the centre of the peg.

## Carrying forward the Meridian observed at the First Station Point.

At B the instrument is again set up and levelled. But here, and at all subsequent stations, the magnetic meridian determined by the compass at the first station is carried forward, and is not reobserved.

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## Field Notes for Fig. 8.

| Station. From | Station. To | Bearing. | Stadia Wires. | Obsd. Dist. | Hor. Dist. | Vertical Circle. | Height of Inst. | Reduced Level of Instrument Collimation. | Line of Sight. Rise +. | Line of Sight. Fall −. | R. L. of Line of Sight at Staff. | Centre Web Reading. | Reduced Level. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A | B | 25° 03′ 00″ | 5·46 / 1·20 | 426 | 426 | ... | ... | ... | ... | ... | ... | 3·33 | ... |
| B | C | 117° 13′ 00″ | 7·36 / 2·96 | 440 | 440 | ... | ... | ... | ... | ... | ... | 5·16 | ... |
| C | D | 217° 33′ 00″ | 6·64 / 4·76 | 388 | 388 | ... | ... | ... | ... | ... | ... | 4·70 | ... |
| D | A | 290° 09′ 00″ | 596 / 239 | 357 | 357 | ... | ... | ... | ... | ... | ... | 4·18 | ... |

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There are two ways of carrying forward the meridian. The first method is to sight the telescope, bubble upwards, from B upon A, with the *back bearing* or direction B–A set on the vernier upon which the forward bearing A–B was read.

A line through 360°–180° on the horizontal circle will then necessarily be exactly parallel with the magnetic meridian adopted at A as the base of horizontal angular measurements, and if the same vernier is set to 360°, the telescope will again point exactly to the north.

The second method is more frequently adopted, although it introduces any instrumental error (see p. 121) which exists owing to want of correct adjustment of the line of sight as denoted by the vertical web of the telescope. It is as follows :—When removing the instrument to the new station, B, leave the horizontal plates clamped, with the original *forward bearing* A–B against the vernier upon which it was read.

When levelling up at B, release the lower clamp H, the upper clamp G being still fast, check the reading of the verniers to see that it has not been accidentally changed, and correct if necessary; this probably will require only a slight touch on the tangent screw. Now release the telescope, reverse it so that the telescope bubble is *beneath* it, and sight back upon A by turning upon the lower axis and using the tangent screw H′ of the lower limb, when the 360°–180° line will be parallel with the magnetic meridian as before.

A moment's thought will show that sighting the *reversed* telescope back from B to A, with the vernier clamped to the forward bearing AB, has the same effect as sighting the *unreversed* telescope along that line with the vernier set to the back bearing. This method can, however, not be adopted except with an instrument which will transit. If a theodolite having stadia lines is used, in which the tele-

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scope cannot be turned right over in its bearings, or in the case of any work requiring high accuracy, it is preferable to adopt the first method.

In either case, when the telescope is directed to the back station it will be found that the compass-needle will automatically come back exactly to the central division (barring local attraction), and will thus prove that no gross error has, so far, been made in observing the bearings. Having sighted on the back station and checked by the compass, align the telescope (bubble uppermost) upon the next forward station, read and book the forward bearing, the centre web and stadia web readings as before, and proceed in exactly the same manner round all the pegs, until the complete circuit has been made, and the bearing AB has been reobserved from A, after a backsight upon the last station, D. Any difference between the bearing AB, as first determined, and the second value for it thus determined, is the angular error of the whole circuit.

All information has then been obtained which is required for the reproduction on paper of the shape and size of the enclosure bounded by the lines AB, BC, CD, and DA, and the calculation of the area they enclose. The above method is called "traversing."

## Fixing Positions by Direct Observation of Rays.

But with a tacheometer, which is capable of measuring both angles and distances, there is a less laborious method of obtaining the desired result in the case of a small enclosure like fig. 8. This consists of determining a ray to each corner from any arbitrarily selected point—which may be one of the station points or may be selected elsewhere as is found convenient. For example, drive in a peg near the middle of the enclosure (fig. 8), set up the instrument over it, and, after levelling up and determining the magnetic meridian,

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read the bearing of and distance to each of the pegs A, B, C, D, in turn.

This will give the direction and length of a series of rays to the corners of A, B, C, D, from whatever point at which the instrument may have been set up.

This method has, however, the disadvantage that no direct measurement of the length of any of the sides is obtained, although it is easy by calculation (see p. 57) to determine both their length and their bearings, if necessary, from the data afforded. It is given here merely to illustrate the fact that from any point at which the instrument is placed the position and, as will be seen later, the "level" or height of any number of other points within range of the telescope, can be ascertained.

It is of course self-evident that there are other methods by which the information obtained by traversing could have been gained, *e.g.* by reading the interior angles formed by the lines A, B, C, D, etc., instead of their bearings, or, again, by measuring the length of one line only, observing the bearings of all the other stations visible from each end of that line, and calculating the sides and angles of the triangles into which the enclosure would thus be divided (triangulation).

But for engineering surveys the traverse method is peculiarly suitable, since in such a survey the straight lines serve merely to connect up a series of points, or "stations," from which rays are observed to those features of the ground of which it is the main object of the survey to record the position and altitude.

Consequently the "stations" are placed at such intervals and in such positions as will facilitate the observation of rays to all desired points, and the passage of such obstacles as forests and buildings by taking advantage of any existing opening.

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## Plotting and Calculation of Areas.

For details of the methods commonly used in plotting surveys, see Chapter VI.

The area enclosed by the lines in fig. 8 can be calculated: (1) with an accuracy about equal to that of the drawing by dividing the figure into triangles, multiplying the base of each triangle by half its vertical height as scaled from the plan, and summing the results; (2) by direct measurement of the plan with a planimeter; or (3) with accuracy by calculation from rectangular co-ordinates, without making the plan at all (see Chapters VI and VIII).

## Determination of Heights.

The question of ascertaining not only the bearings and distances of points observed, but also their heights must now be considered.

Heights are measured either from the mean sea-level as a starting-point or from some imaginary horizontal plane, assumed for the purposes of the survey and referred to as the "Datum Plane" of the survey. The height of a point above the plane thus assumed is called its "Reduced Level" or R.L.

In most countries the heights above mean sea-level of numerous fixed points, called "bench-marks," have been determined and recorded by Government surveys, and these marks therefore form the best starting-points for levelling.

Where this is impossible or inconvenient, it is usual to assume, in round figures, a height above datum, for some well-defined point selected as a "bench-mark" for the survey, and to refer all levels to that point, which may be the highest point on top of, say, a bridge abutment or a church door-step, etc. It must be so selected that it may be readily recognised, and a careful description and sketch of it should be made in the "Remarks" column of the field book for future reference.

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If levels are subsequently taken from such a point to another one of known height above the standard datum plane of the country, it is of course only necessary to add or subtract the difference between its true height thus formed and that assumed for it, to or from all levels on the survey, in order to "reduce" them also to the standard "datum."

## Use of the Tacheometer as an Ordinary Surveyor's Level.

The telescope of a tacheometer, set horizontally, can, of course, be used in exactly the same way and for the same purposes as that of an ordinary surveyor's level, but this is seldom done in practice.

The following example illustrates its use in this manner. In fig. 9 a height of 1000 feet above an imaginary datum line is assumed for the bench-mark A; this value 1000·00 is accordingly the reduced level of A. The heights above datum on this basis, or the reduced levels (R.L.'s) of other points, B, C, D, etc., are required. To obtain them, the staff is held upon A, and the instrument is set up at any place (preferably equidistant approximately from A and B) whence readings can be taken to both stations with the telescope horizontal.

The telescope bubble is brought to the centre of its run, and the centre web reading (say 2·55) is taken upon a staff at A; this reading is entered in the level book (see below) as a *backsight* (since the work is proceeding from A towards D). Because the line of sight of the telescope is horizontal, this value (2·55) is the vertical interval as measured upon the staff, *upwards* from the point A to the collimation line (central line of sight) of the instrument. The R.L. of the collimation line is therefore 1000·00 + 2·55 = 1002·55 feet, in the present position of the instrument.



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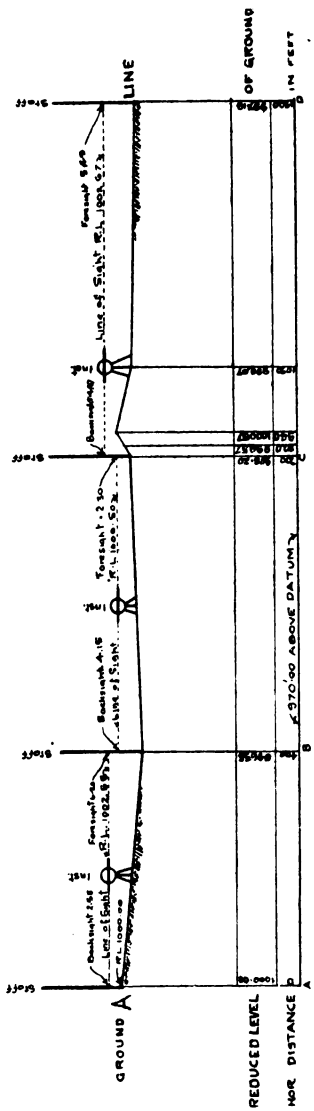


FIG. 9.—Longitudinal Section

The telescope is next directed to the staff at B, and the centre web reading (say, 6·20) on B (still with the telescope horizontal) is entered on the next line of the level book as a "foresight." This foresight *deducted* from the known collimation level of the instrument (1002·55) gives the level of point B (996·35). The instrument is now removed to a point between B and C, and a backsight reading (4·15) is taken upon the staff at B. Adding this to the R.L. of B, the collimation level of the instrument in its new position is found (R.L. 1000·50). The foresight to C (2·30), is deducted from this, and gives 998·20 as the R.L. of C.

In this manner the R.L.'s of all desired points can be determined. Points to which both a foresight and a subsequent backsight are read are called "change points," and the operation of moving the

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instrument to a new position after taking a foresight on, say, B, in order to take a backsight to the same point is commonly called "changing on" B. Before reading a foresight to a change point, any number of "intermediate foresights," or, briefly, "intermediates," can be read to other points whose level it is desired to know.

A special column for these "intermediates" is reserved in the book, as they do not affect the calculation of the collimation level when the instrument is moved from one place to another, but are merely deducted from the collimation level for that "set-up" at which they were taken in order to obtain the R.L.'s of the points to which they refer. Being, usually, merely ground levels, at points not marked by pegs, it suffices to read these intermediates to the nearest tenth of a foot. All backsights, foresights, and observations to bench-marks (which latter should never be taken as intermediates) must be read with the utmost accuracy and with due attention to the level-bubble. For short intermediate sights it does not greatly matter if the bubble is a division or two out of the centre.

## Reducing the Book.

The simple rule for "reducing the book," *i.e.* obtaining the successive R.L.'s of the points observed, is "Add the backsights and deduct the foresights." Starting with the known or assumed reduced level of the first point, add the first backsight to it to obtain the collimation level, and from this collimation level deduct the foresight to the next point to obtain its R.L. The correctness of the work is checked either by continuing to another bench-mark, or point of known R.L., or by levelling back again, along any route to the starting-point. The arithmetical work of reducing the book is checked by totalling all the foresights and all the backsights; the difference between their sums will agree with

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the difference in level between the first and last points observed, if no arithmetical error exists.

LEVEL BOOK.

| Sights. | | | Collimation Level. | Reduced Level. | Station. | Distance. | Remarks. |
|---|---|---|---|---|---|---|---|
| Back. | Inter. | Fore. | | | | | |
| 2·55 | ... | ... | 1002·55 | 1000·00 | A | 00 | ... |
| 4·15 | ... | 6·20 | 1000·50 | 996·35 | B | 400 | ... |
| 4·47 | ... | 2·30 | 1002·67 | 998·20 | C | 900 | ... |
| ... | 3·10 | ... | ... | 999·57 | ... | 920 | ... |
| ... | 2·30 | ... | ... | 1000·37 | ... | 940 | ... |
| ... | 4·60 | ... | ... | 998·07 | ... | 1050 | ... |
| 3 20 | ... | 5·49 | 1000·38 | 997·18 | D | 1500 | ... |
| 8·16 | ... | 1·14 | 1007·40 | 999·24 | ... | ... | Check levels back to A. |
| ... | ... | 7·39 | ... | 1000·01 | ... | ... | Error 0·01 foot. |

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## CHAPTER IV.

### *DETERMINATION OF HEIGHTS AND HORIZONTAL DISTANCES WHEN VERTICAL ANGLES ARE USED.*

In the last chapter a method of using the tacheometer to determine heights was described, corresponding exactly with the procedure used to determine them with an ordinary surveyor's level. But this method of using the telescope as though, like that of an ordinary level, it were incapable of revolution in a vertical plane, has all the disadvantages of working with the latter instrument, and does not take advantage of the special facilities available in a tacheometer. It usually necessitates going twice over the same ground and keeping a separate field book, while in steep country the instrument must be set up at short intervals. While it must be admitted that the accuracy obtainable by the ordinary levelling method exceeds that usually to be expected when heights are calculated from vertical angles and observed distances, and that it is the only method suitable for fixing the levels of bench-marks, yet the vertical-angle method is accurate enough as a basis for preliminary calculations for most engineering purposes. As applied in practice, it has the advantage of much greater rapidity in the field, enabling such a number of points to be fixed both in position and altitude as would be prohibitive in cost were it necessary to fix their height by means of a levelling instrument and their position by chain and theodolite. This advantage



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results, not only in monetary saving, but in the production of a superior plan, since the features of the ground surveyed are delineated from much fuller information. Survey lines should always, if possible, form a closed circuit, thus enabling all errors to be determined; where this is done it will not be necessary to run a second line of check levels. In many cases, however, to close the circuit formed by the traverse lines by making these return to the starting-point, or by finishing at some point whose position and altitude has already been fixed, is impracticable. Under these circumstances, even though the angles and distances be accepted without any further check than the consistent return of the compass-needle to the central division, as described on p. 29, and the distance check obtained by reading all three webs, the levels themselves require checking. This can be done either by reading the backward line as well as the forward one at *each* station in the first instance, or by running a second line of entirely independent levels.

It is quite possible for one surveyor to run both check-levelling instrument and tacheometer together without an assistant, taking the check levels while the tacheometer is being carried between station points; but this double work involves a considerable strain, which renders the result more liable to error, and reduces too much the time available for selecting the station points and the features of the ground which it is desirable to record.

When working single-handed, it is probably better to observe from each station to the one preceding, as well as to the one following it. The results are then all in one field book, and the distances and levels are entirely checked, without going over the ground a second time.

### Effect of Vertical Angles on Stadia Observations.

When stadia readings are taken with the telescope elevated or depressed, so that the line of sight is no longer horizontal,

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allowance must be made for the effect of the vertical angle, in calculating both distances and heights.

It must be remembered that all distances shown upon a plan are *horizontal* distances.

Fig. 10 shows diagrammatically the instrument standing at the foot of an imaginary hill, the graduated staff being held vertically upon the hilltop.

The values to be determined are—

(1) The horizontal distance A–B.

(2) The vertical interval $y$ between the telescope axis and the point where the central line of sight cuts the staff.

With the telescope elevated or depressed from the horizontal, and the staff held vertically, the stadia lines fall upon the latter at points C′, C′, which are farther apart than the points $C^2$, $C^2$ at which the same lines would intersect it if the staff were at right angles to the central line of sight. In this latter position the webs in the telescope are set to subtend the $\frac{1}{100}$th or $\frac{1}{200}$th of the distance from the centre of the instrument to the staff.

Consequently, multiplication of the interval C′C′, as read directly on the staff, by the coefficient K = 100 (or 200) will give too high a value even for the *oblique* distance A–C; it is the smaller interval $C^2C^2$ which, multiplied by the coefficient, gives the true oblique distance AC. And since the angle $\theta'$ at which the staff must be inclined to bring it at right angles to the collimation line AC is evidently equal to $\theta$, as read on the vertical circle, the smaller interval $C^2C^2$ may be taken to be equivalent to (C′, C′) $\cos\theta$ by treating the angle at $C^2$ as a right angle, from which it usually differs so little in practice that the error involved is smaller than the unavoidable error of observation.

## Reduction of Observations involving Vertical Angles.

But in the triangle ABC, which is a right-angled one, the hypotenuse, or the oblique distance AC being K (C′~C′)

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$\cos\theta$, therefore its base AB, or the *horizontal distance*, is

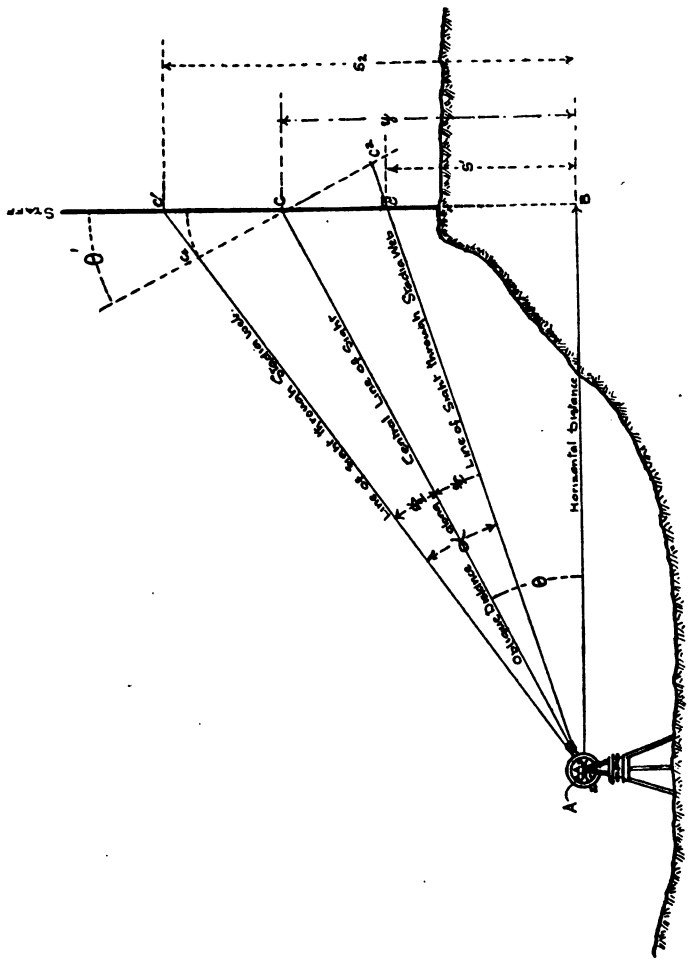


FIG. 10.—Diagram illustrating the effect of Vertical Angles.

$AC \times \cos\theta$ or $K\ (C' \sim C')\ \cos^2\theta$, while the vertical interval $y$ is $AC \cdot \sin\theta$ or $K\ (C' \sim C')\ \sin \cdot \cos\theta$.

In fig. 10, the "measuring angle" $\alpha$, by one-half of which

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$\left(\frac{\alpha}{2}\right)$ the angle at C2 differs from 90°, is greatly exaggerated. If the stadia lines are ruled for a coefficient of 100, the value of $\alpha$ will be 34′ 22″.

The most practical method of performing multiplications by $\cos^2\theta$ and $\sin\theta.\cos\theta$ is to use a special tacheometrical slide-rule (see p. 66), as the results are thereby obtained with great speed and almost entire absence of labour. Books of tables and diagrams for effecting the multiplication are also available, but their use is comparatively slow and laborious.

The exaggeration in the figure shows that, when the line of sight is not horizontal, the centre web reading is not exactly the mean of the stadia web readings. With moderate vertical angles it is sufficiently nearly so for all practical purposes, but the following figures demonstrate that, when the vertical angle is large and the distance great, it is not safe to take the difference between *one* stadia web reading only and that of the centre web, and then use twice the ordinary coefficient, as the resulting *height* may be too greatly affected. In fig. 10 let the measuring angle be 34′ 22″, the distance AB 1000 feet, the vertical angle $\theta = 30°$, and the coefficient 100. Assume twice this coefficient, or 200, for the half measuring angle 17′ 11″.

Then—

| | | | |
|---|---|---|---|
| $S' = AB\tan(\theta-\frac{1}{2}\alpha)$ | $=$ | 570·70 | Diff. 6·65 between central and lower webs. |
| $y = AB\tan\theta$ | $=$ | 577·35 | |
| $S_2 = AB\tan(\theta+\frac{1}{2}\alpha)$ | $=$ | 584·03 | Diff. 6·68 between central and upper webs. |
| Sum of Diffs. | . | | 13·33 between the two stadia webs. |

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From central and lower stadia web only—

$$AC = 6{\cdot}65 \times 200 = 1330 \text{ feet.}$$

From both stadia webs—

$$AC = 13{\cdot}33 \times 100 = 1333 \text{ feet.}$$

From central web and upper stadia web only—

$$AC = 6{\cdot}68 \times 200 = 1336 \text{ feet.}$$

Error from neglecting to read both stadia webs in horizontal distance AB—

$$\pm 3 \text{ feet} \times \cos^2 30^\circ = 2{\cdot}25 \text{ feet.}$$

Error from same cause in height $y$—

$$= \pm 3 \text{ feet} \times \sin . \cos 30^\circ = 1{\cdot}29 \text{ feet.}$$

A short table of the values of $\cos^2 \theta$ and $\sin . \cos \theta$ is given at the end of the book; the interval of 20′ corresponds with that of the vertical circle divisions on most tacheometers.

## Observed Distance, Staff Collimation Level, and Instrument Collimation.

Fig. 11, which is not to scale, shows how the foregoing principles are applied in ascertaining the difference of level and the distance between two points. In what follows, the value of K ( C~C ), which is the product of the length on the staff subtended by the stadia lines and the stadia coefficient (100 or 200, according to the optical arrangement of the telescope), is called the " Observed Distance," the reduced level of the point on the staff intersected by the central line of sight is called the " Staff Collimation Level " to distinguish it from the R.L. of the same line of sight at the instrument, this being the " Instrument Collimation Level " or height of the horizontal axis of the telescope above datum.

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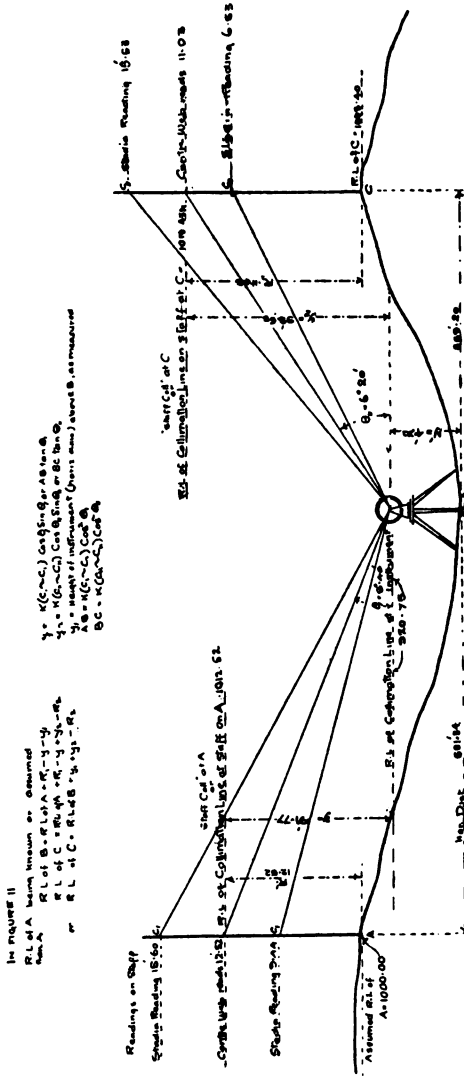

FIG. 11.—Diagram illustrating the use of Vertical Angles in Levelling.

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In fig. 11 the height of the first station, A, is 1000·00 feet above datum. The 1 : 100 stadia wires give the observed distance looking from B to A as $100(15{\cdot}60 - 9{\cdot}44) = 616$ feet. The angle $\theta$ read on the vertical arc is 8° 40′ (a fall from A to B). Therefore the horizontal distance AB is $616 \cos^2 \theta = 616 \times 0{\cdot}977 = 601{\cdot}83$ feet, and the vertical interval $y$ between instrument collimation level and staff collimation level, or the fall of the line of sight from A to B, is $616 \sin . \cos \theta = 616 \times 0{\cdot}14897 = 91{\cdot}77$ feet. By placing a levelling staff against the instrument, or by using a graduated tape as the string of the plumb-bob beneath the instrument, it is found that the height of the telescope axis above B, or, briefly, the "height of instrument" at B, is 4·70 feet. With these data the reduced level of B can be found by addition and subtraction from that of A, as follows :—

| | feet. |
|---|---|
| R.L. of Station A . . . . = | 1000·00 |
| Add reading of centre line . . . | 12·52 |
| Staff Coll. A . . . . | 1012·52 |
| *Deduct* Fall from Staff Coll. at A to Inst. Coll. at B . . . | 91·77 |
| R.L. of Inst. Coll. at B . . | 920·75 |
| *Deduct* Height of Inst. at B . . | 4·70 |
| R.L. of B . . . . = | 916·05 |

Observing from B to C, the observed distance $100 \times (15{\cdot}53 - 6{\cdot}53) = 900$, is obtained. $\theta$, on vertical circle = +6° 20′ (a rise from B to C). Therefore horizontal distance BC $= 900 \times \cos^2 6° 20' = 900 \times 0{\cdot}988 = 889{\cdot}20$ feet.

The vertical interval $y_2$ or rise of the line of sight from B to C, is :—

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| | |
|---|---|
| $900 \times \sin . \cos 6° 20' = 900 \times 0{\cdot}10964 =$ | 98·68 feet. |
| R.L. of Inst. Coll. at B . . | 920·75 |
| *Add* rise from Inst. Coll. to Staff Collimation at C . . . | 98·68 |
| R.L. of Staff Coll. at C . . = | 1019·43 |
| *Deduct* Centre web reading at C . | 11·03 |
| R.L. of C . . . . = | 1008·40 |

The values of $\cos^2$ and sin and cos above need not be taken to a greater number of places than will suffice to give the answer correct to two places of decimals of a foot, even for station points. For ground levels, which are not change points, but which will be used in delineating the natural features of the ground, correctness to the nearest tenth of a foot will suffice.

After a little practice in reducing actual field notes, no necessity will be found for writing down the figures in this manner, as the columns in the field book are specially arranged to facilitate the work of reduction (see p. 24). If at first any doubt be felt whether to add or subtract, a rudimentary sketch will always settle the point.

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## CHAPTER V.

### *THE FIELD WORK OF A CONTOUR SURVEY.*

At the end of the book are given all the field notes of an actual tacheometrical survey for the contour plan shown in Plate I.

The present chapter deals with the field operations of this and other similar surveys; the office work occupies the succeeding Chapters VI and VII. Plate I follows the field notes and is so arranged that it can be pulled out to face any page of the text. It will be seen that from each station both the preceding and following stations were observed; this was done in order to obtain an independent check on the levels, without going over the ground a second time (see p. 37).

### Selecting and Marking Station Points.

It is advisable in selecting instrument stations or traverse points for a survey (especially for one referred to the magnetic meridian, which varies) that the position of at least one or two points be such that they can at any time be found again, without using instruments other than a common measuring-tape, in case it be required to extend or modify the original survey at some future date. On large surveys it is usually practicable to mark at least a few points with masonry beacons; but in ordinary engineering work the



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most that can be done in the way of permanent marks, as a rule, is to drive stout iron stakes flush with the ground at a few points and record the linear measurements to them from conspicuous objects in the immediate neighbourhood. Any kind of pegs, if they project above the ground, are soon knocked out and lost, but iron stakes driven well down can always be found by scraping away the surface soil. For the majority of the station points wooden pegs are used; they soon disappear, but are convenient, cheap, and easily obtained.

If two points of a survey can be recovered, and either the original field notes or the co-ordinates are available, there is no difficulty in re-locating all the others.

The selection of points over which it is difficult to set up an instrument should be avoided. It is also advisable so to select the stations (see Chapter VII *re* Calculation of Distance and Bearing from Co-ordinates) that as many of them as possible may be intervisible; but this consideration is frequently outweighed by that of selecting such points at more or less equal distances apart as will bring the desired features of the country under command of the telescope.

## Length and Arrangement of Traverse Lines.

If the instrument is not set up at points intermediate between the main traverse stations, 500 to 800 feet will be a suitable length for the lines with a telescope of fair quality, fitted with an ordinary stadia diaphragm. But it is advantageous, where the country admits of it, to have such very long straight lines that readings from end to end cannot possibly be taken in one sight, and to fix a number of subsidiary instrument stations at intermediate points on these lines—say, 500 to 600 feet apart,—taking for the length of a line the sum of the distances between all such substations upon it. Observations are taken from the latter to

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fill in all the topography within range of the instrument, which is set up at every alternate subpoint, thus :—

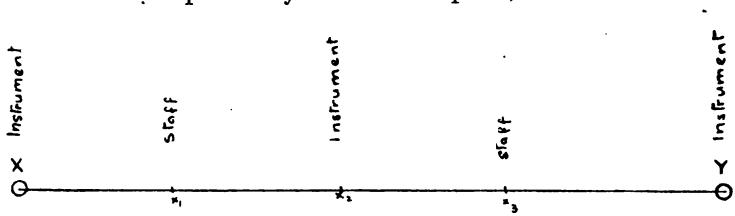


FIG. 12.—Line with Substations.

## Lining in Substations.

When the instrument is at X (fig. 12), the first intermediate point $X_1$ is lined in by means of it; and when all desired sights from X have been taken, a flag is left there and the instrument is removed to $X_2$, which point can be lined in by means of the staff at $X_1$ and the back flag (at X) quite accurately enough to serve the ordinary purposes of such a substation. The adoption of such long straight lines, where possible, tends to reduction of labour in the field and the office, and to increased accuracy, but it is usually practicable only in very open country. This method of arranging the lines would have been a better one for the survey shown in Plate I than that actually adopted. One straight line might have been run from A to E, and another from F to H, making B, C, D, and G all substations on these two straight lines, thus reducing the main traverse to a four-sided figure. It is, however, advisable (*cæteris paribus*) to have the traverse lines more or less equal in length; a line should therefore not be prolonged in such a way as to necessitate its being followed by an unduly short one.

## Flagging Stations.

The usual practice in traverse work is to read the bearings to flags thrust into the ground as close as possible to

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the pegs (instead of having the flags held actually on the centre of each peg), and, in setting up the instrument to centre it over the hole left in the ground by the point of the flag-pole when the latter is pulled out. The exact spot can be marked if necessary by dropping a long nail into the hole; but this is commonly neglected, as the error likely to be introduced if the instrument is on some future occasion set up very slightly to one side of its former position is usually less than the unavoidable error of linear measurement in a line of average length.

## Field Operations for Survey.

In commencing the survey under consideration (Plate I), the initial point A is selected to coincide with the intersection of two farm boundaries, and the second point B is placed on the prolongation of one of these boundaries. Each of these two points is marked by an iron stake driven down flush with the ground, and a flag.

## Location and Description of Initial Line.

A description of the position of A and B having been entered in the field notes, as shown, a third point C is selected and marked (as are subsequent points) with a wooden peg and flag. The instrument is then set over B and levelled up; with the lower axis and compass-needle released, and the horizontal plates clamped together at zero, the whole instrument is turned on the lower axis until the needle comes to rest at the central division of the compass graduation if it is a trough compass, or points to 360° if it is one of circular pattern. The north end of the needle is always observed, as the needle itself may not be quite straight. The lower axis is then clamped and the upper plate released.



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### Bearings observed before taking Stadia Readings.

The bearings from B to flags A and C are next read as described on p. 27 *et seq.*, and are booked as shown in the field notes attached to Plate I. They are found to be 10° 14′ 30″ and 179° 47′ 30″ respectively. Next, the flags are replaced by graduated staves. The telescope bubble is now brought exactly to the centre of its run, and the stadia web and axial web readings are taken to both stations with the bubble in this position, since the levels of A and C with respect to H make it unnecessary to read the vertical angles. On the staff at A the readings are :—Stadia webs, 12·58 and 6·83 ; centre web, 9·70.

### Booking Stadia Readings.

The stadia web readings are booked on the line reserved for station A , and in the fourth column on the field book in the form of a fraction, thus, $\frac{12{\cdot}58}{6{\cdot}83}$ ; the central wire reading goes into the thirteenth column, under its appropriate heading ; while in the seventh, under " Vertical Circle," a cipher is written or a dash made to indicate that the telescope was horizontal. On the line reserved for station C the staff readings to that station are similarly booked.

### Observations completed for Traverse Lines before Observing Topography.

The two most important readings from station B , viz., those of the traverse lines thence to A and C , are the first ones taken after setting up at B , because much trampling of the ground around the tripod feet while observing to numerous topographic points might disturb the instrument. When these readings have been booked, all the topographical points necessary for the purposes of the survey must be recorded ; they will govern the drawing of the contour lines

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on the plan, and the plotting of the general landmarks, fences, buildings, etc.

Not far to the west of point A there is an angle in the boundary of the land under survey, and a consequent change in its direction. A staff is placed at the angle, to which the bearing is 348° 44′, and, with the telescope horizontal, the cross webs read :—Stadia, 7·15 and 1·15 ; central, 4·15.

These readings are therefore booked in their proper columns, all on the same line in the field book, while in the "Remarks" column a note is made, "At angle in South boundary of farm Fairview."

To fix the direction of the fence beyond the angle, the staff is now moved to a new position further along it. But since this point is at a higher level than the telescope, the vertical circle must be used.

### Method of Using the Vertical Circle.

After releasing its clamp, and elevating the telescope by hand sufficiently to bring all the webs to bear upon the staff, it is reclamped, and with a half-turn or so of the tangent screw, the nearest even division on the vertical circle is brought to the vernier zero. The bearing of the staff, 335° 20′, and the readings on it—stadia, $\frac{14{\cdot}00}{5{\cdot}40}$, and central, 9·70—are taken and booked as before, and the angle set on the vertical arc is also booked in the appropriate column and marked + or − according as it happens to be an angle of elevation or depression. In this case it is +1° 00′.

### Cross Hair should not be set to an Even Foot on the Staff.

Some surveyors, in reading elevated or depressed sights, set one of the cross hairs by means of the tangent screw to an even foot on the staff, and book whatever angle is then

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found on the vertical circle. There is nothing gained by this. The *other* cross hairs will not be at an even foot on the staff, while the vertical circle vernier has to be read and booked to odd minutes and seconds, thus involving more figures, and making the reduction to horizontal distances and difference of level more troublesome. Cases, of course, arise in which it is not practicable to set the vertical circle at an even division, but they are infrequent, since the readings of the central web and *one* stadia web are all that is absolutely necessary as a minimum, and they can be taken upon any part of the staff which may happen to be visible.

## Advantages of Setting to an Even Division on Vertical Circle.

Making the vertical angle correspond with an even division on the circle has the following advantages :—

1. It is unnecessary to read minutes and seconds on the vernier—a half-turn of the tangent screw will usually bring the nearest division into line with its zero, thus saving time and eye-strain.

2. There are fewer figures to book.

3. The same vertical angles recur frequently, the whole of a day's work, perhaps, only involving angles varying by half-degrees or whole degrees from $\pm 0°$ to $\pm 5°$ or thereabouts.

Consequently much time and labour are saved in making calculations—if a slide-rule is used, the readings fall upon even divisions of the $\cos^2$ and sin . cos scales ; while if tables are used, very simple and short ones suffice, and there is no need for interpolation to odd minutes and seconds.

Examination of the field notes referring to Plate I will show that in using the vertical circle for forty-one different sights, only eight different angles are used, and that with one exception—40′—they are all multiples of a half-degree.

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And out of the forty-one sights, the angle 1° recurs twelve and the angle 1° 30′ eleven times.

The work of looking out $\cos^2$, and sin . cos for thirty-three other angles is thus avoided, even in so short an example as this ; while twenty-three readings, or more than 50 per cent., can be reduced with the same two settings of a slide-rule.

## Topographic Points.

Next, two or three readings east and west of the line A–B are taken, to determine the slope of the ground in this direction. If there were any marked changes in this slope, similar readings would be taken at each place where a change occurs, but in this case there are none ; the points for these "spot-levels" are so selected as to give a cross section more or less at right angles to A–B, and about halfway between A and B . These are the points whose bearings in the notes are 315° 00′, 348° 30′, and 20° 45′, and whose levels on the plan are 1013·82, 1007·60, and 1001·10. It is not necessary to read the bearings of spot-levels of this kind to single minutes. They will be plotted by protractor, from accurately fixed station points, and, therefore, it is quite sufficient to record the whole degrees and the fraction of a degree as estimated by eye, without actually reading the vernier, as must be done for bearings to station points and other landmarks.

Next the top of the river bank is fixed and the water level where the farm boundary crosses the river, as is also the point where there is a change of direction in the river bank between A and B, noting what these points are in the "Remarks" column. This completes the work from Station B .

## Care required in taking Water and Station Levels.

Water levels require considerable exactitude of measurement, and as it is impossible to read them with the telescope horizontal, it is as well before reading them with the

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telescope pointed in the required direction, to bring the vernier of the vertical circle to the horizontal diameter of the latter and see that the telescope bubble is exactly in the centre of its run. If it is not so, bring it back by a very slight movement of one footscrew before setting the vertical angle. A division or two of eccentricity in the position of the telescope bubble makes little or no practical difference in fixing most topographical points or features of the ground, as the resulting errors are inappreciable on plan, but water levels and station points are quite a different matter. That is why the reading to the forward station C was taken immediately after levelling up the instrument and reading back to A, and before having, by trampling the ground round the tripod in taking subsidiary readings off the traverse line, risked a slight disturbance or settlement of the instrument.

It is always worth while, when reading to a station point with a vertical angle, and after pointing the telescope at the staff, to see that its bubble is central when the vertical circle vernier reads zero, and, if not, to make it so with one footscrew as described.

### Double Observations to Bench-marks.

When vertical angles are read to a bench-mark, two readings should be taken: one with the telescope bubble in its usual position, and one with the telescope reversed and the bubble underneath. The mean of these two will be free from errors of adjustment in the instrument.

Having finished at station B, the instrument is moved to C, and a staff and flag is left at B, while a new forward station D is selected.

### Carrying forward the Meridian of the Survey. Backsight to Preceding Station.

At C, after levelling up and setting the bearing B–C, 179° 47′ 30″ on the horizontal plate, the telescope is reversed

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so that the bubble is underneath, and sighted on the flag at B by revolving the instrument on the *lower* axis, leaving the plates clamped. The compass-needle thereupon comes back automatically to its central division (unless local attraction exists), and shows that no gross error has been made in the previous bearing B–C. Readings on the staff at B are next taken, with the *telescope still reversed*, thus obtaining a second determination of the line BC. Since, when reading from B to C, the telescope bubble was on top, while in reading from C to B it is underneath the telescope, the mean of these two sets of staff-readings will be free from adjustment errors of the horizontal line of sight. The telescope is now turned over so that the bubble is again on top, the forward readings to D are taken, and are followed by all desired readings to points within range of the telescope.

### Use of Substation.

The instrument is then removed to D, a new forward station E is selected and flagged, and a similar routine is performed. The line EF is rather long for convenient reading, while from E, owing to the intervention of the high river bank, a staff held in the stream bed at water-level cannot easily be observed.

A substation E′ is therefore ranged in upon the river bank exactly on the line EF, and after finishing at E, the instrument is set up at E′. The sum of the lengths E–E′ and E′F gives the whole length EF, and from E′ the required detail of the river-bed can be picked up.

### Check Bearings to Station and Other Points.

At F and the ensuing stations G and H the work is similar to that at B, C, and D, but from F across to D, and from G to the stations A, B, C, and D additional bearings are taken. These points are intervisible, and as the lines joining them form a number of triangles which give a check on the work,

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enabling the length of a line in any of the triangles to be calculated upon the basis of the measurement of another line either in the same triangle or in any other forming one of a connected series.

In unclosed traverses, where there are no convenient intervisible stations for the observation of check bearings, it is often possible to observe to some conspicuous object, such as a church spire, from three or more stations; this affords a very valuable check on an otherwise unclosed traverse, which should never be neglected if the opportunity for obtaining it occurs. (See Chapter VII.)

### Field Check on Total Error in Bearings.

The instrument is finally set up at A, and a bearing of the first line AB is taken after backsighting on the last station H. Any difference between the value thus obtained for bearing AB, and the value originally determined for it is the total angular error in all the traverse lines.

### Measurement of Angles of Deflection instead of Bearings.

Some surveyors prefer, instead of reading the actual bearings directly in the field, to record the angle to the right or left through which each line deflects from the preceding one. This method is not quite so convenient as the "azimuth method" described in the foregoing pages, and the compass check on the angular work is not obtained at each station, nor the field check by actual re-observation of the bearing of the first line as determined from that obtained for the last one. It lends itself well, however, to the elimination of adjustment errors in the instrument, by taking the mean of two determinations of the angle, the second one being made with the telescope inverted, and to exact measurement of angles by repetition, as follows :—

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### Repetition of Angles.

With the vernier set at zero, the telescope is directed to the first point, the lower axis is clamped, and the angle to the second point read in the usual way. Then by turning on the lower axis, the telescope is redirected to the first point with the first angle read still set on the plates, the lower axis is reclamped, and the telescope again turned round the upper axis to the second point; the vernier reading is then the sum of two determinations of the angle. The telescope is again brought back to the first point by means of the lower axis, and the operation continued in a similar manner, until the sum of four or five readings is shown on the vernier. Supposing that, after four repetitions with an instrument reading to single minutes, the vernier showed 348° 27′, then the true value of the angle observed is one-fourth of this, or 87° 06′ 45″. An even number of observations should be made, using the telescope alternately in its normal position (bubble up) and inverted (bubble down). This repetition method is, however, only used when very precise values are required—seldom or never in traverse work, except in mining and tunnel surveys underground.

### Check on Deflection Angles and Interior Angles.

If deflection angles and not bearings are measured, then the check on the angular work of a closed traverse is that the sum of the angles to the right should differ from that of those to the left by exactly 360°. In a closed traverse, if the interior angles are measured or calculated, the sum of these angles should be equal to twice as many right angles as the figure formed by the traverse has sides, less four right angles.

### Solution of a Triangle.

The following example shows the solution of the triangle A, B, G, in which the data are the length AB and the bearings of all three lines. (Plate I and field notes.) The angle

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formed by two lines meeting at a point is simply the difference between their bearings to or *from* that point. In Plate I, if the length GB (not observed in the field) is required, the angles of the triangle ABG are obtained as follows :—

AB = 575 feet.

| | | |
|---|---|---|
| Bearing A–G | = 148° 32′ 30″ | ( = Bearing GA −180°) |
| ,, A–B | = 190° 14′ 30″ | ( = Bearing BA +180°) |
| Angle A . | = 41° 42′ 00″ | |
| Bearing BA | = 10° 14′ 30″ | |
| ,, BG | = 121° 22′ 00″ | ( = Bearing GB −180°) |
| Angle B . | = 111° 07′ 30″ | |
| Bearing GA | = 328° 32′ 30″ | |
| ,, GB | = 301° 22′ 00″ | |
| Angle G . | = 27° 10′ 30″ | |
| ,, A . | = 41° 42′ 00″ | |
| ,, B . | = 111° 07′ 30″ | |
| Sum . . | = 180° 00′ 00″ | |

The sides of triangles are proportionate to the sines of the angles opposite to them ; therefore

$$GB : AB : : \sin A : \sin G,$$

$$\text{or } GB = \frac{\sin A}{\sin G} \times AB = \frac{(0{\cdot}6650131)}{(0{\cdot}4567098)} \times 575 = 837{\cdot}52 \text{ ft.}$$

| | | |
|---|---|---|
| or by logarithms, log sin A | = | 9·8229721 |
| *Add* log sin AB | = | 2·7596678 |
| Log of AB sin A = Sum | = | 12·5826399 |
| *Deduct* log sin G | = | 9·6596403 |
| Log GB | = | 2·9229996 |

The number of this log or GB = 837·52 feet as before.

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### Solution of Same Triangle by Slide-Rule.

To check either of the above solutions of a triangle by the slide-rule :—Set 27° 10′ on the sine scale of slide to 575 on top fixed scale, and over 41° 42′ on sine scale read 838 on top scale.

This is an example of the great utility of a slide-rule in survey work; the accuracy of result, 838 feet instead of 837·52, is greater than the accuracy of the drawing, and hence is amply good enough for checking the length GB as plotted, while the laborious multiplication by natural sines, or the use of logarithms, is replaced by a mechanical operation occupying no more time than the mere looking out of only the *first* log required.

### Compass Surveys with the Tacheometer.

For preliminary purposes, surveys and reconnaissances can be made, with sufficient approximation to the truth, by means of a prismatic or other compass, and a passometer—levels being taken either barometrically or run in long sights with a levelling instrument. In such work the bearings are determined to about $\frac{1}{4}$ degree by the compass, reading from a fresh magnetic meridian at each station, as indicated by the needle, while distances are obtained by pacing.

### Superiority of Tacheometric Methods in Compass Surveys.

Work of this class can be done with about equal *rapidity* with the tacheometer, especially if staves are used which have a more open graduation than usual. The general principles of the work are much the same as those already described, but the arrangement of the instrument and staff stations is somewhat different, nor are so many checks upon the work obtained.

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Much *greater accuracy* can be maintained, and far more detail can be picked up than is possible with the level, compass, and passometer, or level, compass, and rough chaining method.

## Method of Working.

At each "set-up" of the instrument it is merely revolved about the lower axis until the compass-needle points to zero of its graduation, care being taken, in the case of a compass fitted to the *upper limb* instead of in its usual position below the horizontal plates, that the horizontal circle is first set

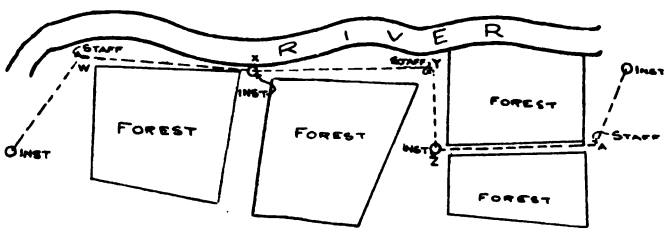


FIG. 13.—Arrangement of Stations in Compass Survey.

to zero (360°). The lower axis is then clamped, the upper one released, and the telescope, always bubble upwards, is directed to the desired points, and the usual readings taken and booked. Local attraction can be detected by setting up at each station, in order that the bearing of each line may be read in both directions; but if that be done, the surveyor might just as well carry forward the same meridian throughout, with accuracy, by sighting back with the telescope reversed, and little would be lost in point of speed. The instrument is therefore usually oriented by the compass-needle alone and is set up only at *alternate* stations. Suppose W, X, Y, Z, etc., in fig. 13 above, to be traverse stations of a compass survey.

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### Instrument Stations.

In order to determine these lines by compass, it is only necessary to set up the instrument at X and Z, bring the compass-needle to zero, and take from X a backsight to the staff at W and a foresight to that at Y; from Z a backsight to Y and a foresight to A, and continue in this manner. By reading all three cross hairs a check is obtained on each distance, and practically all the closing error is angular; corrections to make the traverse close should, therefore, be applied principally to the bearings.

### Reading Compass Bearings.

Only when and where necessary, in order to secure details of topography, is the instrument set up over a staff point, as at Y, in which case the addition to the R.L. of Y of the measured height above that point of the instrument as set up, gives the R.L. of Inst. collimation at Y. In purely compass work it is useless to read the bearings more closely than to the nearest 15′ or quarter degree, as the compass usually cannot be relied on even to this extent of accuracy.

The saving of time is chiefly due to the fact that the instrument is only set up at alternate stations, so that double the usual distance is covered at each set-up, and to the avoidance of delay in selecting and flagging stations, as the observations are made to the staves themselves and flags are dispensed with.

As soon as the back sight to W from X has been read, the staff-holder at W holds his staff at any intermediate points required for spot-levels between W and Y; he then comes forward ready to accompany the surveyor to Z, whence he is sent to point A, while observations from Z to Y and around these points are being taken on the other staff or staves.

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The number of staves used depends largely on the speed of the observer at the instrument. It is advantageous for a quick observer to have six, eight, or even more, staff-holders in order to avoid loss of time in carrying the staves from point to point when picking up topography. Two of the staff-holders are then used for station points only, and flags are sometimes tied to their two staves to distinguish them from the others.

The figures are booked exactly as is done in the field notes to Plate I for the first set-up at B, reading back to stations A and forward to C.

## Common Mistakes in Surveying Topography.

One of the commonest mistakes of beginners in tacheometry consists in the observation of too many spot-levels. Unless for a part of the plan—say, the site for a bridge or some such work, which will eventually be much enlarged to show close detail,—there is nothing whatever gained by plastering the ground with innumerable spot-levels, many of which represent nothing in particular.

The aim should be to secure all the really *representative* points, including such levels on side-long ground as will show the true general slopes, indicate their changes, and enable the correct interpolation of contour lines. It is a waste of time to record detail which cannot be plotted to the scale of the plan, and which would in any case have no effect on the engineering project to be based on that plan. Field books are often seen containing fifty to a hundred spot-levels taken from a single instrument station, when the same information, and the same result on plan, could have been gained from, say, ten or twelve, if properly selected. This simply means that five or six times the necessary money has been spent on the survey work On engineering surveys the critera in deciding to observe a point or disregard it should

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be: "Will it affect the project in any way?" "Can it be shown on the plan?"

### Adaptability of the Tacheometer.

The tacheometer can be so used as to yield any desired degree of accuracy in the results; the greater the accuracy and the fuller the detail, the slower the work will be and the more it will cost. The methods of working should therefore so be varied that the results obtained are always a little better than is necessary for the purpose in view, and no more.

### Reduction of Field Notes.

In the field, the columns of the field book headed "Stations, From and To," "Bearing," "Stadia," "Vertical Circle," "Height of Instrument," and "Central Wire," have all been filled in.

### Reduction of Stadia Readings to Observed Distance.

The next step is to calculate from these figures the distances and levels of all the points observed, and the arrangement of the columns in the field book enables this to be done in a logical sequence and in a more or less mechanical manner. The first proceeding is to enter throughout the book, in the column headed "Observed Distance," the difference of the two stadia readings as read by the stadia wires.

### Reduction of Observed Distances to Horizontal.

Having done this, each "Observed Distance" is, if necessary, multiplied by the square of the cosine of the vertical angle entered against it in the column headed "Vertical Circle," and the results are written into the "Horizontal Distance" column, which is next in order to that for "Observed Distance."

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## Cases in which Reduction to Horizontal can be neglected.

A little consideration will, however, show that this reduction to the horizontal can be disregarded in many instances, the observed distance being used without reduction.

For, up to a vertical angle of 5° 40′, the difference is under 1 per cent., while for an angle of 4° it is only about 0·5 per cent., and for an angle of 2° it is approximately 0·1 per cent. Taking the average length of sights to be 500 feet, and remembering that the *staff* can only be read at that distance to the nearest $\frac{1}{100}$th of a foot (equal to 1 foot of distance), the length read as 500 feet may in actuality be 499·5 or 500·5 feet. Multiplication by $\cos^2 2°$ would give 499·5 feet, instead of 500—a result which, in view of the possible error in the original staff reading, makes no improvement in point of accuracy.

Moreover, in the case of observations taken to fix spot-levels which are not on the traverse line, nothing is gained by greater accuracy in measurement of distance than can be detected by the use of the scale with which these points are plotted on the plan. It is hardly possible to measure less than $\frac{1}{100}$th of an inch on a plan with an ordinary drawing scale, and even at the comparatively large scale of 200 feet to the inch this $\frac{1}{100}$th represents 2 feet; on one of 500 feet to the inch it is 5 feet.

Assuming the length of an average sight to be as great as 1000 feet (which is never the case), it is practically impossible to appreciate the difference as plotted on plan between the "Observed Distance" and the "Horizontal Distance" due to a sight inclined at the following vertical angles:—

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1° | on a plan drawn to scale | | | 100 | feet | = | 1 inch. |
| 2° | ,, | ,, | ,, | 200 | ,, | = | 1 ,, |
| 3° | ,, | ,, | ,, | 300 | ,, | = | 1 ,, |
| 4° | ,, | ,, | ,, | 400 | ,, | = | 1 ,, |
| 5° | ,, | ,, | ,, | 500 | ,, | = | 1 ,, |

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As a rough guide, it may therefore be said that distances which are taken to fix spot-levels usually need not be reduced to the horizontal unless the vertical angle contains a greater number of degrees than there are hundreds of feet to the inch on the plan, so that in cases where the vertical angle is of equal or less degree, the column for "Horizontal Distance" can be left blank and the "Observed Distance" used when plotting.

### Extent of Accuracy necessary in Reduction to Horizontal.

Where it is necessary to reduce the distances to the horizontal, there is no gain in making use of a greater number of decimals in the value of $\cos^2$ than will give the result correct to the nearest six inches, unless the staff is read more closely than to the nearest $\frac{1}{100}$th of a foot in the field.

### Calculation of Rise or Fall of Line of Sight.

When the reduction to the horizontal has been made, where necessary, the columns for "Rise or Fall of Line of Sight" are filled in, by multiplying the observed distance (not the *horizontal* distance) by $\sin . \cos \theta$, the angle $\theta$ being the angle entered opposite to each observed distance in the "Vertical Circle" column. The results are entered under "Rise" or "Fall" according as the vertical angle is marked + or −. This calculation must always be made, and, unlike the reduction to the horizontal, *cannot be neglected where the vertical angle is small.*

### Use of Sin $\theta$ instead of Sin × Cos $\theta$.

But, for 2°, the difference between $\sin . \cos \theta$ and $\sin \theta$ is only sufficient to make a difference of $\frac{1}{100}$th foot of level in 1000 feet, so that $\sin \theta$ can, if desired, be substituted for



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$\sin . \cos \theta$ with vertical angles up to 2°, in connection with distances not exceeding 1000 feet, and up to 4° with distances of about 500 feet or less. Values of "Distance $\times \sin \theta$" can be found in ordinary traverse tables in the columns headed "Departure"; but the use of such tables or of direct multiplication is clumsy and laborious compared to the use of a proper tacheometric slide-rule.

### Special Slide-Rules for Reduction of Field Book.

Messrs T. Cooke & Sons, of Westminster, make a small and inexpensive stadia slide-rule which can easily be carried in the pocket; and the "Universal" slide-rule, which is larger and heavier, and with which all ordinary calculations can be made, is now obtainable to suit graduations of either 360° or 400 grades.

These rules have scales of $\cos^2 \theta$ and $\sin . \cos \theta$, and their use, which can be acquired by half an hour's practice, renders the labour of reduction for vertical angles almost negligible. (See Chapter IX.)

### Calculation of Reduced Levels for all Station Points precedes that for any Topographical Points.

The "Rise or Fall of Line of Sight" having been filled in throughout, the next operation is to calculate the levels of the instrument stations (and bench-marks, if any) in order to check the work before proceeding to make the same calculation in regard to the numerous spot-levels. As this reduction is sometimes a little confusing, owing to the complications introduced by the rise and fall of the line of sight, it is given below in full, for the field notes to Plate I. After a little practice, no difficulty will be found in making the necessary additions and subtractions mentally, and writing the results at once in their proper columns.

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## Detailed Example: Calculation of R.L.'s of Station Points in Field Notes for Plate I.

| | Feet. |
|---|---|
| *From B.*—Starting with the level of A, which is assumed to be . . . . . R.L. | 1000·00 |
| The centre wire reading from B on the staff at A, with the vertical arc at zero, was . . | 9·70 |
| Therefore the collimation level of the instrument when set up at B is . . . . | 1009·70 |
| The centre wire reading on C, with vertical arc at zero was . . . . . | 5·11 |
| Therefore the Reduced Level of C is. . . | 1004·59 |
| And from the "Height of Instrument" column it is seen that, at B, the axis of the instrument was 4·65 above the point B itself; therefore the level of B is 1009·70 − 4·65 = 1005·05. | |
| *From C.*—Level of B . . . . . | 1005·05 |
| When the instrument was at C, the C.W. reading on B (vertical arc at zero) was . . | 4·80 |
| Therefore collimation level at C . . . | 1009·85 |
| (Check the level at C by deducting the "Height of Instrument" from the collimation level at C, thus:—1009·85 − 5·26 = 1004·59.) | |
| Rise of line of sight from C to D . . . | 8·55 |
| Therefore R.L. of line of sight at staff on D . | 1018·40 |
| Centre wire reading on staff at D . . . | 3·42 |
| Therefore the Reduced Level of D . . . | 1014·98 |

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| | Feet. |
|---|---|
| *From D.*—R.L. of point C . . . . . | 1004·59 |
| Centre wire reading on C . . . . . | 6·64 |
| R.L. of line of sight, at staff on C . . . | 1011·23 |
| Fall of line of sight from D to C, *i.e. rise* of line of sight from C to D . . . . | 8·55 |
| R.L. of instrument collimation at D . . . | 1019·78 |
| (R.L. of D = 1019·78 − 4·80 = 1014·98, to check.) | |
| Rise of line of sight from D to E . . . | 7·50 |
| R.L. of line of sight at staff on E . . . . | 1027·28 |
| Centre wire reading on E . . . . . | 7·25 |
| Therefore R.L. of E . . . . . | 1020·03 |
| *From E.*—R.L. of point D . . . . . | 1014·98 |
| Centre wire reading on D (vertical arc at zero) . | 9·62 |
| R.L. of instrument collimation at E . . . | 1024·60 |
| (R.L. of E = 1024·60 − 4·57 = 1020·03.) | |
| Fall of line of sight D to E′ . . . . | 17·80 |
| R.L. of line of sight at staff on E′ . . . | 1006·80 |
| Centre wire reading on E′ . . . . . | 12·20 |
| R.L. of E′ . . . . . | 994·60 |

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| | Feet. |
|---|---|
| *From E'*.—R.L. of point E . . . . . | 1020·03 |
| Centre wire reading on E . . . . . | 7·11 |
| R.L. of line of sight at staff on E . . . | 1027·14 |
| Rise of line of sight E′ to E, *i.e.* fall of line of sight. . . . . | 27·82 |
| R.L. of instrument collimation at E′ . . . | 999·32 |
| (R.L. of E′ = 999·32 − 4·72 = 994·60.) | |
| Rise or fall of line of sight E′ to F . . . | 0·00 |
| R.L. of line of sight on staff at F . . . | 999·32 |
| Centre wire reading on staff at F . . . | 2·62 |
| R.L. of F . . . . . | 996·70 |
| *From F*.—R.L. of E′ . . . . . | 994·60 |
| Centre wire reading on staff at E′ . . . | 6·80 |
| R.L. of line of sight on staff at E′ . . . | 1001·40 |
| Rise or fall of line of sight F to E′ (vertical arc zero) | 0·00 |
| Instrument collimation at F . . . . | 1001·40 |
| (R.L. of F = 1001·40 − 4·70 = 996·70.) | |
| Rise of line of sight F to G . . . . | 10·50 |
| R.L. of line of sight on staff at G . . . | 1011·90 |
| Centre wire reading at G . . . . . | 7·50 |
| R.L. of G . . . . . | 100·40 |

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| | Feet. |
|---|---|
| *From G.*—R.L. of F | 996·70 |
| Centre wire reading on staff at F | 12·41 |
| Rise or fall of line of sight G to F (vertical arc zero) | 0·00 |
| Instrument collimation at G | 1009·11 |
| (R.L. of G = 1009·11 − 4·71 = 1004·40.) | |
| Rise or fall of line of sight G to H | 00·0 |
| R.L. of line of sight on staff at H | 1009·11 |
| Centre wire reading on staff at H | 4·05 |
| R.L. of H | 1005·06 |
| *From H.*—R.L. of G | 1004·40 |
| Centre wire reading on staff at G (vertical arc zero) | 5·46 |
| R.L. of line of sight on staff at G | 1009·86 |
| Rise or fall of line of sight H to G (vertical arc zero) | 0·00 |
| R.L. of instrument | 1009·86 |
| (R.L. of H = 1009·86 − 4·80 = 1005·06.) | |
| Rise or fall of line of sight H to A | 0·00 |
| R.L. of line of sight on staff at A | 1009·86 |
| Centre wire reading on staff at A | 9·84 |
| R.L. of A | 1000·02 |
| True value of R L. of A | 1000·00 |
| Error | 0·02 |

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| | Feet. |
|---|---|
| *From* A.—R.L. of H . . . . . | 1005·05 |
| Centre wire reading on staff at H . . . | 7·14 |
| R.L. of staff collimation at H . . . . | 1012·19 |
| Fall of line of sight H to A . . . . | 7·47 |
| R.L. of instrument collimation at A . . . | 1004·72 |
| (R.L. of A, 1004·72 −4·70 = 1000·02.) | |
| Rise of line of sight A to B . . . . | 5·01 |
| R.L. of staff collimation at B . . . . | 1009·73 |
| Centre wire reading on staff at B . . . | 4·69 |
| R.L. of B . . . . . | 1005·04 |

The "R.L. of Instrument Collimation" and "R.L. of Staff Collimation" and "Reduced Level," are all entered in the field book, opposite the stations to which they refer, and in the columns allotted to them. It will be seen that there are two distinct determinations of the level of each station, and of the length of each line—in fact, there are six values for the length of every line, found by multiplying either the difference between the stadia readings by the coefficient, or the difference between the centre wire reading and that of either stadia wire by double the ordinary coefficient, although the latter course is less accurate, since a difference of $\frac{1}{100}$th foot in the interval read on the staff will then represent 2 feet of distance instead of 1 foot. If, after completing the reduction for all station points up to the control point (the starting-point itself in a closed circuit), a serious discrepancy is disclosed between the R.L. of the latter as then found and its correct R.L., it is easy to locate the mistake, strike out the erroneous entry in the field notes, and adopt the alternative observation, remembering that an

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error in distance may be the cause of an error in height owing to the use of the vertical arc.

The bearings are checked in the field, by comparison of the bearing originally found for the first line, and its bearing as determined from a backsight on the last station.

## Calculation of Reduced Levels of Topographical Points.

The R.L.'s of the spot-levels or "Intermediates" are calculated last of all. The R.L. of instrument collimation is, of course, a constant for each "set-up" of the instrument. Add to or deduct from its value at any station the "rise or fall of line of sight" (+ if a rise, − if a fall) for each intermediate sight taken from that station, entering the resulting figures in the column of "R.L. of Line of Sight at Staff" or "Staff Collimation Levels." From these latter figures the centre wire readings are always to be deducted to obtain the reduced levels of the topographical points to which the intermediate sights were observed.

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## CHAPTER VI.

### *PLOTTING.*

THE two methods of plotting surveyed points in common use are :—

(1) By means of a protractor and scale ;

(2) By rectangular co-ordinates for the station points, the details being afterwards plotted from these by means of a protractor and scale.

### Comparison of Common Methods.

The first method avoids the calculations involved in the second, but is less accurate, and gives no check on the work until the whole traverse has been plotted and the starting-point has again been reached.

The second method is highly accurate: the closing error of the work can be determined exactly before plotting, and the necessary corrections can be applied ; moreover, the exact direction and distance between any points co-ordinated can be determined from the figures, so that the co-ordinates form a second record of the position of the station points, in case the field book be lost or destroyed. The exact area of any rectilinear figure can be calculated from the co-ordinates of its corners, and the work of plotting can be divided among several draughtsmen, with the certainty that the various sheets of the plan will fit together as desired.

The co-ordinates of the station points should, therefore,



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always be calculated, unless the work be of very limited extent and no great accuracy is necessary.

## Plotting by Protractor.

Before commencing to plot, it is advisable to sketch out the traverse lines roughly on a piece of tracing-paper, in order to facilitate the selection of such a position for the starting-point and such a direction for the meridian or north and south line as will result in the whole work falling in to a good position on the sheet. If this be neglected, or if no thought be given to the general trend of the traverse as to direction, it may be found that the lines run off the paper.

A common, and wrong, method of proceeding is to select a point on the paper for the first station of the survey, draw a north and south line through it, prick off from this the bearing of the first traverse line, draw a line from the starting-point along this bearing, and scale off the required distance along the line thus drawn to determine the second station. Through this second station a new north and south line is drawn, and the process repeated for each of the following stations.

This is a most inaccurate way of using the protractor, for it involves the delineation of a fresh meridian on the paper at each station point; such meridians, as drawn, may or may not be exactly parallel.

## Correction of Closing Errors.

The proper way is to use a fair-sized protractor, say 8-inch to 10-inch diameter, and to prick off the bearings of all the lines from a meridian drawn either through the first station point or elsewhere on any convenient part of the sheet. The bearings thus laid down are then transferred to the desired points by running them across with a rolling parallel ruler as the work proceeds.

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Even so, it is most improbable that the traverse as drawn will return exactly to the starting-point. If not, the plotting must be checked, and if it be found correct, corrections must be applied as follows :—Suppose fig. 14 represents the traverse as plotted. The side H′A′, which should close in again to the starting-point at A, finishes at A′, showing a closing error equal to the length A′–A. Join A′–A, and draw a line parallel to it in the direction of A′–A through each of the other corners.

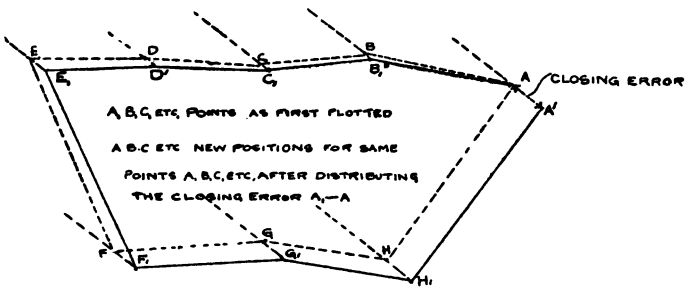


FIG. 14.—Distribution of Closing Error.

Then as the

$$\left.\begin{matrix}\text{Sum of the}\\ \text{length of}\\ \text{all sides}\end{matrix}\right\} : \left\{\begin{matrix}\text{Length of}\\ \text{A′–A, or}\\ \text{total error}\end{matrix}\right\} :: \left\{\begin{matrix}\text{Length of}\\ \text{first line}\\ \text{AB′}\end{matrix}\right\} : \left\{\begin{matrix}\text{Correc-}\\ \text{tion for}\\ \text{point B′.}\end{matrix}\right.$$

Lay off the correction thus found along the line parallel to A′–A through B′; this fixes the corrected position of B.

At point C we have—

$$\left.\begin{matrix}\text{Sum of}\\ \text{all sides}\end{matrix}\right\} : \left\{\begin{matrix}\text{Total}\\ \text{error}\end{matrix}\right\} :: \left\{\begin{matrix}\text{Sum of}\\ \text{sides AB′}\\ \text{and B′C′}\end{matrix}\right\} : \left\{\begin{matrix}\text{Correction}\\ \text{for point C′.}\end{matrix}\right.$$

Lay off this correction from C′ along the line through the latter parallel to A′–A, and proceed in a similar manner for the other points.

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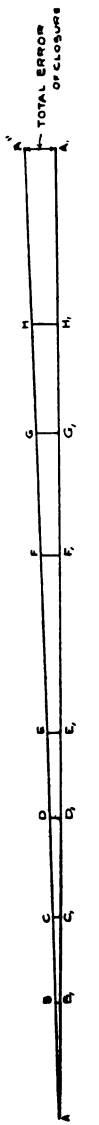


FIG. 15.—Graphic Apportionment of Closing Error.

A graphic method of ascertaining the corrections is as follows:—To a convenient scale, draw a line A–A′ (fig. 15) equal to the sum of all the sides, and set off the lengths of the successive sides along it, as AB′, B′C′, etc.

Draw a line perpendicular to A–A′ through A′ to A″, and make its length equal to the total closing error; join A–A″, and erect perpendiculars through the points B′, C′, D′, etc., cutting the line AA″ at B, C, D. The lengths B′–B, C′–C now represent, on the scale to which the figure is drawn, the corrections to be set off as before at the points B, C, D, etc.

## Plotting Spot-Levels and Details.

The usual method of plotting the spot-levels, by drawing a meridian line through each traverse point, laying off the required bearings from it with the protractor, and then pricking off the distances to each point by means of an ordinary boxwood or ivory scale, is both slow and clumsy. The constant handling of the separate scale and protractor wastes much time. A far better plan is as follows:—Make a blue print from an ordinary transparent circular protractor; if it has the degrees figured in both directions, first blacking out with a band of Indian ink the figures which run from 0 to 360 in the direction of the hands of a clock. This blue print will then have the degrees numbered only in the *reverse* direction. Trim the blue print off to an even circle, and if the paper on which it is made is weak, back it with a thickness of

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drawing-paper. It is of course preferable to use mounted ferro-prussiate paper in the first instance, but this is not always available.

Then cut an opening along the 360° line as shown in fig. 16, and gum a thin strip of card, or of stout paper,

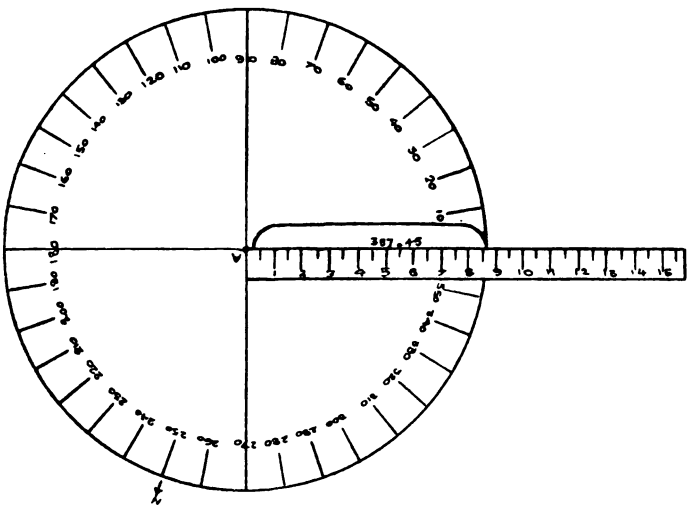

FIG. 16.—Special Protractor for plotting Spot-levels.

along this line. Make a fine pinhole at the centre of the protractor, and, starting from this as zero, graduate the strip to the same scale as that of the plan required.

## Advantages of Special Protractor.

This protractor is used as follows :—

A fine needle, with a lump of sealing-wax to form a head, is passed through the central pinhole and pressed into the paper at the station point whence it is desired to plot, so that it acts as a vertical axis about which the protractor can be revolved. At the edge of the latter, a small pencil

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mark is made on the paper to indicate a north line through the station point. To lay off a given bearing and distance, the *bearing* is brought to the pencilled North mark by revolving the protractor about the needle, and the distance is at once pricked off along the strip which carries the scale. In the figure the position of a spot-level, 387·45 R.L., is shown plotted on bearing 250°, at distance 550 feet from A. The scale can be extended to any desired length, so long as it will remain straight, for plotting distances greater than the radius of the protractor, while distances less than the radius are marked off through the opening cut for that purpose. The figures of level value are then written in at once, using the dot which indicates their position as the decimal point.

It will be seen that for each point the protractor is merely revolved to bring the bearing to the North mark,—there is no continual picking up of scales, no readjustment of the protractor over the station, no figures of bearings pencilled in around its edge, and no extension of the lines beyond their exact length. It is possible, by this means, to plot spot-levels almost as quickly as the figures can be read out by an assistant.

Several variations of this protractor are in use; some are made of celluloid with interchangeable scales, and others of brass, semicircular in shape, with scales along the diameter line and graduations in both directions; but none are better or quicker in use than the home-made blue print protractor described above. Stout tracing-paper protractors, figured in the reverse direction as described, can be obtained from T. Cooke & Sons of Westminster; with the addition of a paper scale, and an opening cut with the scissors, they are excellent, and cost a few pence only. It is as well to stick a wafer at the centre, in order to form a strong bearing for the needle.

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### Elaborate Protractors.

For the accurate protracting of the station lines it is possible to purchase a metal protractor fitted with verniers reading to single minutes. Such instruments are, however, delicate, costly, slow in use, and rarely seen in any practical drawing office. It is far better, in the case of work requiring such a degree of accuracy, to plot by the system of rectangular co-ordinates described below.

### Rectangular Co-ordinates.

In fig. 17 (*a*), let $\theta$, the bearing from North of the line GH, be between 0° and 90°, like the sixth line of the traverse on Plate I. Draw a North and South line, NGS, through G, and from H draw HN perpendicular to GN and meeting it at point N. The line GH now forms the hypotenuse of a right-angled triangle, having GN for base and NH for perpendicular. The length GN is the distance from G to H measured along the meridian, or the projection of the line GH upon the meridian line; this is called the "difference of latitude" between G and H. The length NH is the distance by which H departs from the meridian drawn through G, or its difference of departure from G. GN and NH are the rectangular co-ordinates of the point H as measured from G. By elementary trigonometry the difference of latitude, GN, is $GH \cos \theta$, and the difference of departure, NH, is $GH \sin \theta$. Using these values, the position of H in relation to G can be plotted without a protractor by measuring $GH \cos \theta$ along the meridian towards the north from G, and from the point so found setting off $GH \sin \theta$ perpendicular to, and to the east of, the meridian.

Next, take the case of a line DE, having a bearing $\theta$ between 90° and 180° (*i.e.* South-east), like the fourth line in the traverse on Plate I. A line S-E (fig. 17 (*b*)) perpendicular to the meridian NS, and passing through E, completes

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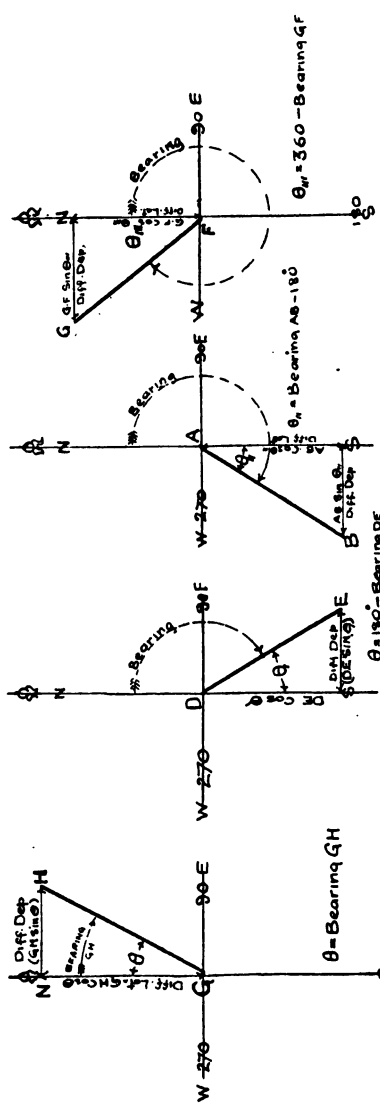


FIG. 17.—Reduction of Bearings for Calculation of Rectangular Co-ordinates.

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the right-angled triangle DSE, in which DS is the difference in latitude, and SE that in departure of E from D, and the angle SDE is $180° - \theta$. Accordingly, the co-ordinates of E from D are—

$$\text{Diff. of Latitude} = \text{DE} \cos (180° - \theta').$$
$$\text{Diff. of Departure} = \text{DE} \sin (180° - \theta').$$

When a line has a bearing between 180° and 270° (*i.e.* South-west), the angle required (BAS in fig. 17 (*c*)) is "Bearing minus 180°"; if the bearing is between 270° and 360°, as in fig. 17 (*d*), then "360° – Bearing" gives the angle GFN.

In every case the angle of which the cosine and sine are multiplied by the length of a traverse line to find the difference of latitude and departure between its two ends, is the smallest angle which that line makes with an imaginary meridian through the starting-point, and is found thus :—

| | | |
|---|---|---|
| Bearing between | 0° and 90°, | use bearing unaltered. |
| ,, ,, | 90° and 180°, | use 180° – Bearing. |
| ,, ,, | 180° and 270°, | use Bearing – 180°. |
| ,, ,, | 270° and 360°, | use 360° – Bearing. |

The angles thus formed are called the "reduced bearings" of the lines to which they refer. Of course if a line bears exactly 90°, or 270°, there is no difference of latitude between its two ends, while their difference of departure is the whole length of the line; if it bears exactly 360° or 180°, the difference of latitude is the whole length of the line, and there is no difference of departure.

### Direction of Co-ordinates. Use of + and – Signs.

The *directions* of the differences of latitude, North or South from the starting-point, and of departure, East or West from the meridian, are indicated by the signs + (plus) and – (minus) placed before the co-ordinates, according to the



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direction of the original (*not* the *reduced*) bearing of each line. Examine a circular protractor, placed so that 360° is at the top, and consider the direction of a line starting from its centre and passing thence through any degree between 0 and 90°—that is, any line whose *bearing* is between 0 and 90°. Such lines run *upwards* and to the *right*. The difference of latitude (length × cosine) for such a line

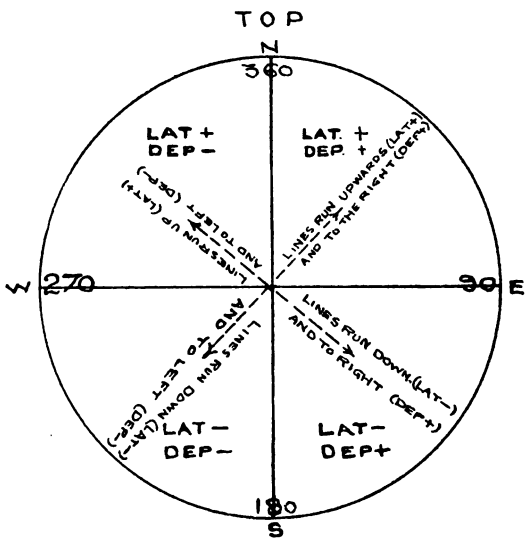


FIG. 18.—Signs indicative of Direction, for Rectangular Co-ordinates.

is + because the line tends upwards, and the difference of departure is + because the line goes to the *right*.

For a line whose bearing is in the next quadrant, between 90° and 180°, the diff. lat. is − because the line tends *downwards*, while the diff. dep. is still + because the line runs to the right.

Similar considerations apply to the other two quadrants, so that the signs to be used are, as shown in fig. 18 :—

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| | | Diff. Lat. | Diff. Dep. |
|---|---|---|---|
| Bearing between | 0° and 90° | + | + |
| ,, ,, | 90° and 180 | − | + |
| ,, ,, | 180° and 270° | − | − |
| ,, ,, | 270° and 360° | + | − |

### Total Co-ordinate Values measured from Initial Point.

After the differences of latitude and departure for each line have been calculated and given their proper signs, they are added together algebraically up to each station point to obtain its co-ordinates, as measured from the initial point of the survey, which usually is either assumed as zero, both in latitude and departure, or has *positive* values in both directions assumed for it, sufficiently large to avoid having any points on the survey with negative co-ordinates,—the latter proceeding being convenient when areas are to be calculated from the co-ordinates.

### Office Methods of Calculation.

The values of $\cos\theta \times$distance and $\sin\theta \times$distance can be obtained by arithmetical or logarithmic computation, or by the aid of a calculating machine, using ordinary mathematical tables in either case. But probably the simplest and least laborious method is to take the values directly from good traverse tables.

R. L. Gurden's tables, published by Griffin & Co., are very suitable, and of ample scope. In them are given, under the headings of "Cosine" and "Sine," values of latitude and departure for every minute of arc by single feet to 100 feet of distance, calculated to four places of decimals, so that values can be correctly ascertained for a line 10,000 feet long to the nearest eighth of an inch approximately, by moving the decimal point two places to the right.

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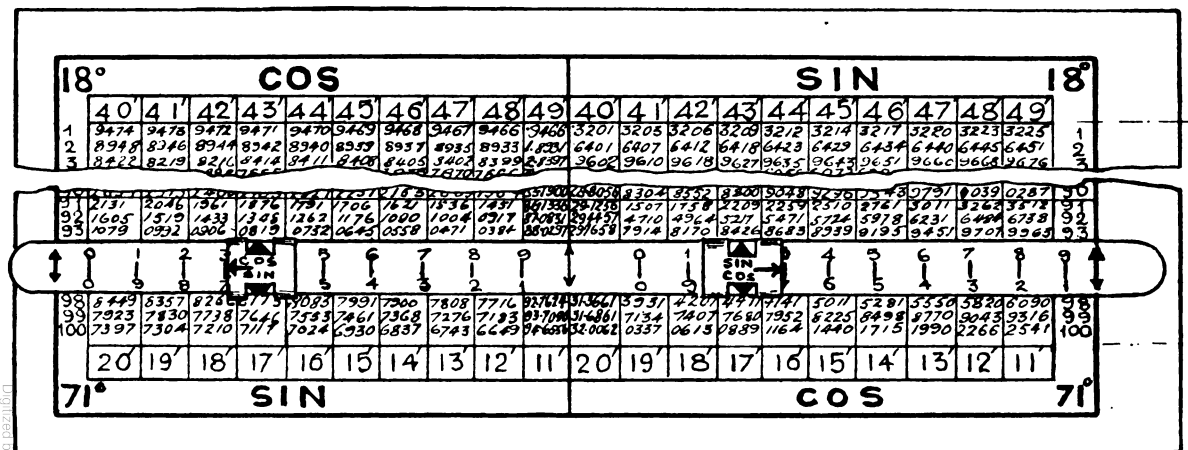


FIG. 19.—Pointer for use with Traverse Tables.

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## Device to Facilitate Readings of Traverse Tables.

To avoid the fatigue of following down the long columns of figures by eye, the following device is a great convenience when using such tables. Two sliding pointers are fitted to a flat ruler or slip of wood, as sketched above in fig. 19. The latitude and departure being printed side by side on the same page with the degrees and minutes of arc at the top of each column, the pointers are moved to make their distance apart correspond with that of the desired columns, and the slip is then moved down the page until the edge of the ruler coincides with the distance sought, when the pointers indicate the required values. In order that the rule may be kept in position laterally, there are marks upon it near the ends which coincide with the border lines of the table. These sliding indicators are not, as far as the writer knows, obtainable ready made—his own, which has been in use for many years, was made in camp from an old ivory paper-cutter, with pointers cut from a tobacco-tin, and answers the purpose admirably.

## Simple Mechanical Device for Calculating or Checking Co-ordinates.

For compass surveys, trial line surveys, for many small scale maps, and other such purposes, values of $\cos\theta \times$ distance and $\sin\theta \times$ distance can be obtained by using a diagram similar to that shown in fig. 20, which is mentioned in Wellington's *Railway Location*, and was originally due to Mr Chas. Francis of the Mexican Central Railway.

This diagram can be made upon a sheet of cross-section paper, with divisions of any desired closeness, the scale selected for the sides (latitudes) and top and bottom (departures) is also drawn upon a straight-edge pivoted to the bottom left-hand corner, as shown. Upon the top and right-hand edge, angles from 0° to 90° are indicated, as on

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a protractor. On swinging the rule to the appropriate reduced bearing $\theta$ of a line and pricking off along it the length L of that line, values of $L \cos \theta$ and $L \sin \theta$ are read at once upon the diagram. The accuracy of the results is limited to the size and subdivision of the diagram used; if, for instance, the scale is 10 feet = 1 inch (subdivided to

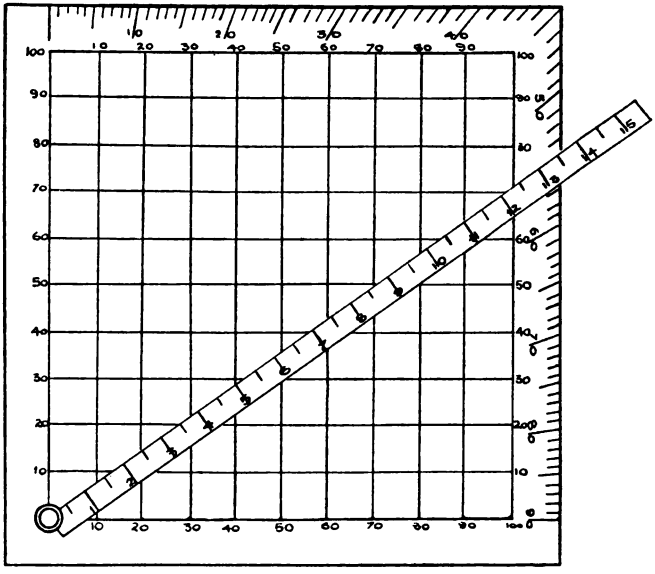

FIG. 20.—Diagram for Calculation of Co-ordinates.

tenths), two figures can be read and a third closely estimated, thus giving results for lines up to 1000 feet long correct within a foot or so.

The diagram is also useful for roughly checking the rigorous computations of co-ordinates. It is evident that, if desired, a similar one can be made in which the rule is centrally pivoted, and the degrees are carried right round the circle (see fig. 21), each quarter of the square inscribed

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in the circle being graduated upon the lines of the small square shown in fig. 20. With such a diagram the bearings can be used as they stand, without reduction, and the signs + and − can be taken from the diagram itself.

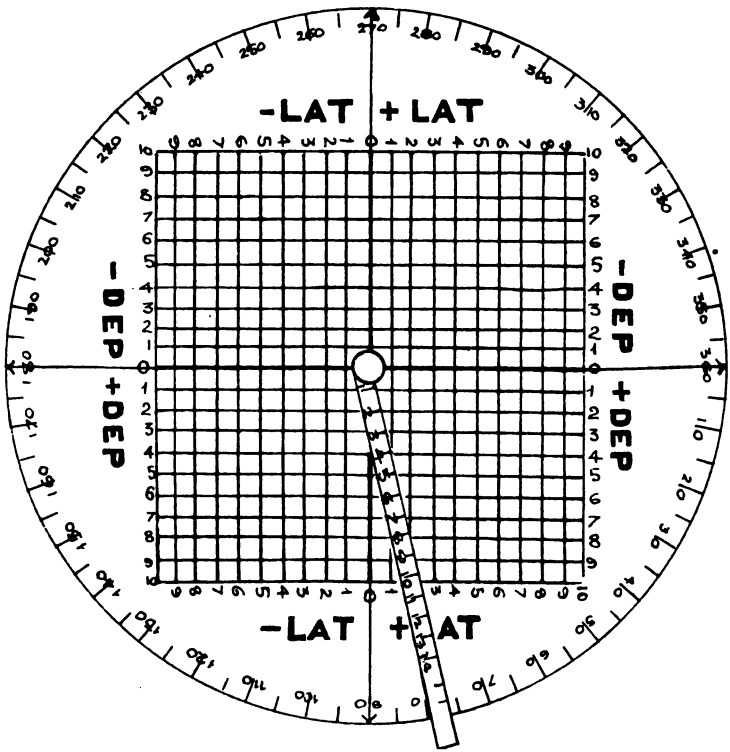


FIG. 21.—Circular Diagram giving Values and Signs of Co-ordinates.

To illustrate the working out of co-ordinates, they are here calculated for the station points of the survey shown in Plate I, first by logarithms and then by traverse tables. The advantage of the latter is very apparent. The bearings and distances are taken from the field notes.

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## Example of Calculation of Co-ordinates from Field Notes (Plate I).

| Bearing and Distance. | Latitudes (Cos). | Departures (Sin). |
|---|---|---|
| A. | O. | O. |
| A to B | *By Logs.* | |
| ° ′ ″ | Log 575 . . 2·7596678 | Log 575 . . 2·7596678 |
| 190 14 30 | Log Cos . . 9·9930245 | Log Sin . . 9·2504376 |
| −180 | | |
| 10 14 30 | Log diff. Lat. . 2·7526923 | Log diff. Dep. . 2·0101054 |
| 575 feet. | Diff. Lat. . . −565·84 | Diff. Dep. . . −102·23 |
| | Add Co-ordinates of A . . ± 0·00 | Add Co-ordinates of A . . 0·00 |
| | B = −565·84 | B = −102·23 |
| | *By Tables.** | |
| | For 570′ . . 560·918* | For 570′ . . 101·346 |
| | For 5′ . . 4·920 | For 5′ . . 0·889 |
| | Diff. Lat. . . −565·838 | Diff. Dep. . . −102·235 |
| | Add from A . ± 0·000 | Add from A . ∓ 0·000 |
| | B = −565·838 | B = −102·235 |
| B to C | *By Logs.* | |
| ° ′ ″ | Log 414 . . 2·6170003 | Log 414 . . 2·6170003 |
| 180 | Log Cos . . 9·9999976 | Log Sin . . 7·5602874 |
| 179 47 30 | | |
| 0 12 30 | Log diff. Lat. . 2·6169979 | Log diff. Dep. . 0·1772877 |
| 414 feet. | Diff. Lat. . . −414·00 | Diff. Dep. . . + 1·50 |
| | Add from B . − 565 84 | Add from B . − 102·23 |
| | C = −979·84 | C = −100·73 |
| | *By Tables.* | |
| | For 410′ . . 409·997 | For 410′ . . 1·490 |
| | For 4′ . . 3·999 | For 4′ . . 014 |
| | Diff. Lat. . . −413·996 | Diff. Dep. . . + 1·504 |
| | Add from B . − 565·838 | Add from B . − 102·235 |
| | C = −979·834 | C = −100·731 |

* One opening of the tables only as compared with five by the logarithmic method —a saving of 80 per cent. in time.

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| Bearing and Distance. | Latitudes (Cos). | | Departures (Sin). | |
|---|---|---|---|---|
| A. | O. | | O. | |
| C to D | *By Logs.* | | | |
| 181° 10′ 00″ | Log 490 . . | 2·6901961 | Log 490 . . | 2·6901961 |
| −180 | Log Cos . . | 9·9999100 | Log Sin . . | 8·3087941 |
| 1 10 00 | Log diff. Lat. . | 2·6901061 | Log diff. Dep. . | 0·9989902 |
| 490 feet. | Diff. Lat. . . | − 489·90 | Diff. Sin . . | − 9·98 |
| | Add from C . | − 979·84 | Add from C . | − 100·73 |
| | D = | −1469·74 | D = | − 110·71 |
| | *By Tables.* | | | |
| | For 490′ . . | − 489·900 | For 490′ . . | − 9·977 |
| | Add from C . | − 979·834 | Add from C . | − 100·731 |
| | D = | −1469·734 | D = | − 110·708 |
| D to E | *By Logs.* | | | |
| 180° ′ ″ | Log 430 . . | 2·6334685 | Log 430 . . | 2·6334685 |
| 179 15 30 | Log Cos . . | 9·9999636 | Log Sin . . | 8·1120465 |
| 0 44 30 | Log diff. Lat. . | 2·6334321 | Log diff. Dep. . | 0·7455150 |
| 430 feet. | Diff. Lat. . . | − 429·96 | Diff. Dep. . . | + 5·57 |
| | Add from D . | −1469·74 | Add from D . | − 110·71 |
| | E = | −1899·70 | E = | − 105·14 |
| | *By Tables.* | | | |
| | For 430′ . . | − 429·964 | For 430′ . . | + 5·566 |
| | Add from D . | −1469·734 | Add from D . | − 110·708 |
| | E = | −1899·698 | E = | − 105·142 |

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| Bearing and Distance. | Latitudes (Cos). | | Departures (Sin). | |
|---|---|---|---|---|
| A. | | O. | | O. |
| E to F | | *By Logs.* | | |
| 69° 35′ 0″ | Log 872 . . | 2·9405165 | Log 872 . . | 2·9405165 |
| 872 feet. | Log Cos . . | 9·5426321 | Log Sin . . | 9·9718233 |
| | Log diff. Lat. . | 2·4831486 | Log diff. Dep. . | 2·9123398 |
| | Diff. Lat. . . | + 304·19 | Diff. Dep. . . | +817·22 |
| | Add from E . . | −1899·70 | Add from E . | −105·14 |
| | F = | −1595·51 | F = | +712·08 |
| | | *By Tables.* | | |
| | For 870 . . | 303·495 | For 870 . . | 815·347 |
| | For 2 . . | ·697 | For 2 . . | 1·874 |
| | Diff. Lat. . | + 304·192 | Diff. Dep. . | +817·221 |
| | Add from E . | −1899·698 | Add from E . | −105·142 |
| | F = | −1595·506 | F = | +712·079 |
| F to G | | *By Logs.* | | |
| 360° 0′ 0″ | Log 602 . . | 2·7795965 | Log 602 . . | 2·7795965 |
| 350 30 30 | Log Cos . . | 9·9940132 | Log Sin . . | 9·2172314 |
| 9 29 30 | Log diff. Lat. . | 2·7736097 | Log diff. Dep. . | 1·9968279 |
| 602 feet. | Diff. Lat. . . | + 593·76 | Diff. Dep. . . | − 99·27 |
| | From F . . | −1595·51 | From F . . | +712·08 |
| | G = | −1001·75 | G = | +612·81 |
| | | *By Tables.* | | |
| | For 600′ . . | 591·785 | For 600′ . . | 98·942 |
| | For 2′ . . | 1·973 | For 2′ . . | ·329 |
| | Diff. Lat. . | + 593·758 | Diff. Dep. . | − 99·271 |
| | From F . . | −1595·506 | From F . . | +712·079 |
| | G = | −1001·748 | G = | +612·808 |

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| Bearing and Distance. | Latitudes (Cos). | | Departures (Sin). | |
|---|---|---|---|---|
| A. | O. | | O. | |
| G to H | | *By Logs.* | | |
| 13° 45′ 30″ | Log 622 . . | 2·7937904 | Log 622 . . | 2·7937904 |
| 622 feet. | Log Cos . . | 9·9873567 | Log Sin . . | 9·3762614 |
| | Log diff. Lat. . | 2·7811471 | Log diff. Dep. . | 2·1700518 |
| | Diff. Lat. . . | + 604·15 | Diff. Dep. . . | +147·93 |
| | From G . . | −1001·75 | From G . . | +612·81 |
| | H = | − 397·60 | H = | +760·74 |
| | | *By Tables.* | | |
| | For 620′ . . | 602·210 | For 620′ . . | 147·452 |
| | For 2′ . . | 1·942 | For 2′ . . | ·475 |
| | Diff. Lat. . . | + 604·152 | Diff. Dep. . . | +147·927 |
| | From G . . | −1001·748 | From G . . | +612·808 |
| | H = | − 397·596 | H = | +760·735 |
| H to A | | *By Logs.* | | |
| 297° 37′ 00″ | Log 858 . . | 2·9334873 | Log 858 . . | 2·9334873 |
| 360 | Log Cos . . | 9·6661001 | Log Sin . . | 9·9474674 |
| 62 23 00 | Log diff. Lat. . | 2·5995874 | Log diff. Dep. . | 2·8809547 |
| 858 feet. | Diff. Lat. . . | +397·73 | Diff. Dep. . . | −760·25 |
| | From H . . | −397·60 | From H . . | +760·74 |
| | Closing Error to A = | + 0·13 | Closing Error to A . . | + 0·49 |
| | | *By Tables.* | | |
| | For 850′ . . | 394·020 | For 850′ . . | 753·158 |
| | For 8′ . . | 3·710 | For 8′ . . | 7·088 |
| | Diff. Lat. . . | +397·730 | Diff. Dep. . . | −760·246 |
| | From H . . | −397·596 | From H . . | +760·735 |
| | Closing Error to A | + 0·134 | Closing Error to A . . | + 0·489 |

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## Distribution of Errors of Closure.

In a closed traverse, if the survey and the calculations were made with absolute accuracy, the sum of the plus $\left\{\begin{matrix}\text{latitudes}\\\text{departures}\end{matrix}\right\}$ would be exactly equal to that of the minus $\left\{\begin{matrix}\text{latitudes}\\\text{departures}\end{matrix}\right\}$. If the closing error is very small, as in the foregoing example, its correction may for most practical purposes be disregarded, or it may at once be divided between one or two lines only. The general principle governing the distribution of errors when the angular and linear measurements may be expected to have about the same degree of accuracy is :—

$$\left.\begin{matrix}\text{Sum of}\\\text{lengths}\\\text{of all}\\\text{sides}\end{matrix}\right\} : \left\{\begin{matrix}\text{Length}\\\text{of any}\\\text{given}\\\text{side}\end{matrix}\right\} :: \left\{\begin{matrix}\text{Total}\\\text{error}\\\text{of lat.}\\\text{or dep.}\end{matrix}\right\} : \left\{\begin{matrix}\text{Correction for}\\\text{lat. or dep. of}\\\text{given side.}\end{matrix}\right.$$

But if the angular measurements probably surpass the linear ones in point of accuracy, use the formula :—

$$\left.\begin{matrix}\text{Sum of}\\\text{all lats.}\\\text{and deps.}\\\text{without}\\\text{regard to}\\\text{algebraic}\\\text{signs.}\end{matrix}\right\} : \left\{\begin{matrix}\text{Length}\\\text{of any}\\\text{given}\\\text{side}\end{matrix}\right\} :: \left\{\begin{matrix}\text{Total}\\\text{error}\\\text{of lat.}\\\text{or dep.}\end{matrix}\right\} : \left\{\begin{matrix}\text{Correction}\\\text{for lat. or}\\\text{dep. of}\\\text{given side.}\end{matrix}\right.$$

In neither case is the error eliminated ; it is merely allotted in reasonable proportions to the various lines of the traverse.

## Plotting by Co-ordinates.

It facilitates the work of plotting if the paper is divided into squares of convenient size by lines parallel to the direction adopted for the meridian, crossed by others at right angles, the side of a square representing some such even length as 500 feet, 1000 feet, or 2000 feet, according to

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the scale of the plan. One of the north and south lines is then selected for the zero line of departures, and one of the east and west lines for that of latitudes; the intersection of these two lines gives the position of the initial point of the survey.

### Selection of Position for Zero Lines.

The selection of the zero line on the paper is made by reference to the co-ordinate values—the greatest plus value in the latitudes being the greatest distance of any station point to the north of the zero line of latitudes, and the greatest minus value the distance to the South of the most southerly point. Similarly with the departures—the point with the greatest plus value is the most easterly and that with the greatest minus value the most westerly from the zero line of departures. In the co-ordinates just worked out (p. 88 *et seq.*), the greatest plus and minus values in latitude are $\pm 0$ (station A), and $-1899$ feet (station E). There is, therefore, no station to the *North* of the starting-point A, and room must be allowed on the paper for at least four squares of 500 feet sides, or 2000 feet, to the south of A, so that E will fall on the sheet (Plate I). This approximately fixes the position of the zero line for latitudes, for which a line is selected, allowing a reasonable amount of space on the paper to the North of A and to the South of E, as there may be spot-levels taken North or South from both points. In the same way the zero line for departures is selected. Here the maxima are for H, which has a departure from zero of $+760$, *i.e.* is 760 feet east of the starting-point, and for D, which is $-110$, or 110 feet to the West of that point; any line can be selected for the zero line of departure which leaves sufficient room on the paper for these stations and the spot-levels taken from them. The intersection of the two zero lines fixes point A, since its co-ordinates are zero in both directions.

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## Numbering Co-ordinate Lines.

The zero lines are marked ±0 at each end. The Latitude lines on the north side of zero are marked successively +500, +1000, +1500, etc., or +1000, +2000, and so on, according to the size of the squares. Those on the south side are similarly marked, but with the minus sign. The Departure lines are marked in the same way, those east of zero being plus and those to the west minus.

The *direction* on the paper of the lines forming the co-ordinate squares need not be parallel with the edges of the sheet. It is convenient to make it so, especially with small plans, but in many cases the work will not fall within the limits of the paper unless the squares are placed diagonally upon the latter. It is usual to draw the squares lightly in pencil, only inking in the intersections of the lines near the edges of the plan and erasing the remainder when the work is finished. The co-ordinates of the various station points are set off with a scale from the co-ordinate lines nearest to their values thus (see Plate I). The latitude of B is −565·8, and its departure is −102·235. It is, therefore, 65·8 feet away in the minus (South) direction from the −500 latitude line and 102·2 feet in the minus (West) direction away from the zero departure line. From the intersection of these two lines, scale off 65·8 feet along the line running south, and 102·2 feet along that running West; the intersection of perpendiculars from the points thus found is the position of B, and a line joining A and B will, if tested with a protractor and scale, be found to have exactly the required direction and length. The other station points are plotted in a similar manner.

## Filling in Topography.

When all the station points are plotted, the spot-levels and other topographical details are filled in from them with a protractor, as described on p. 78.

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## Contours.

On plans for engineering purposes after plotting the traverse lines and spot-levels, the " contour lines " or " contours " are filled in from the information so recorded.

## Nature of Contour Lines.

These contours are simply lines drawn on the plan in such a way that all points through which they pass are at the same height above datum, *i.e.* they all have the same reduced level. Imagine a pond full of still water, with a line traced around it exactly at the water's edge. This would be a contour line, traced full size on the ground. Suppose that the water level in the pond then sank by 1 foot, and again a line was traced around its edge. This would be a second contour line, at 1 foot vertical interval from the first one traced. On a plan, which represents the ground itself to a small scale, there are no such water lines to guide the tracing of contours, but there are instead the actual levels (spot-levels) of a large number of points, and between any two points thus fixed there can be found by simple proportion the position of other points at any desired intermediate height. The slope of the ground is assumed to be uniform between the points whose levels are recorded in the field notes. Hence the great importance of recording *all changes* in the slope of the ground within the limits of possible deviation in alignment of a project. Every contour line, or line of equal altitude, must close upon itself somewhere, but not necessarily within the limits of the map. Contours delineating small depressions and summits will probably close within those limits, but the lines indicating the general slopes of the country will usually run to the edge of the plan, unless the survey is extensive and the scale small.

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## Vertical Interval between Contours.

Contours are usually drawn at equal vertical intervals and numbered with the reduced level they represent, the amount of this interval being determined by the purpose of the plan. Five- or ten-foot vertical intervals are common on engineering general plans, to be used for the study and rough location of possible projects; but at the probable sites of works of construction, such as bridges, dam-walls, buildings, etc., an extra amount of information is recorded in the field, and the contours are frequently drawn at 1-foot or 2-foot intervals. The reason for this is obvious: variations in the ground surface which might be of absolutely no practical importance if they occurred, say, in the basin of a huge reservoir, may be quite another matter if within the area which will be covered by the masonry of the dam-wall itself. Contour lines, by reason of their nature, have the following general characteristics (see fig. 22):—

## Characteristics of Contour Lines.

A contour forming a closed circuit, and having no other contour line within that circuit, is either very nearly a summit or very nearly the lowest level of a depression; the reduced level of the actual summit or of the lowest point within the closed contour is usually shown by a dot with the height written against it. Comparison with the values written against the adjacent contours shows whether a summit or depression is indicated by such a closed contour, as at (A), a summit, and (B), a depression. The contours of a uniform slope are uniformly spaced. The steeper the slope, the closer the contours, and *vice versa*.

Watershed lines and stream lines always cross contours at right angles: see C and D, which are correct, and E and F, which are impossible. Contours can only touch one another in the case of a vertical drop, such as a cliff or river bank

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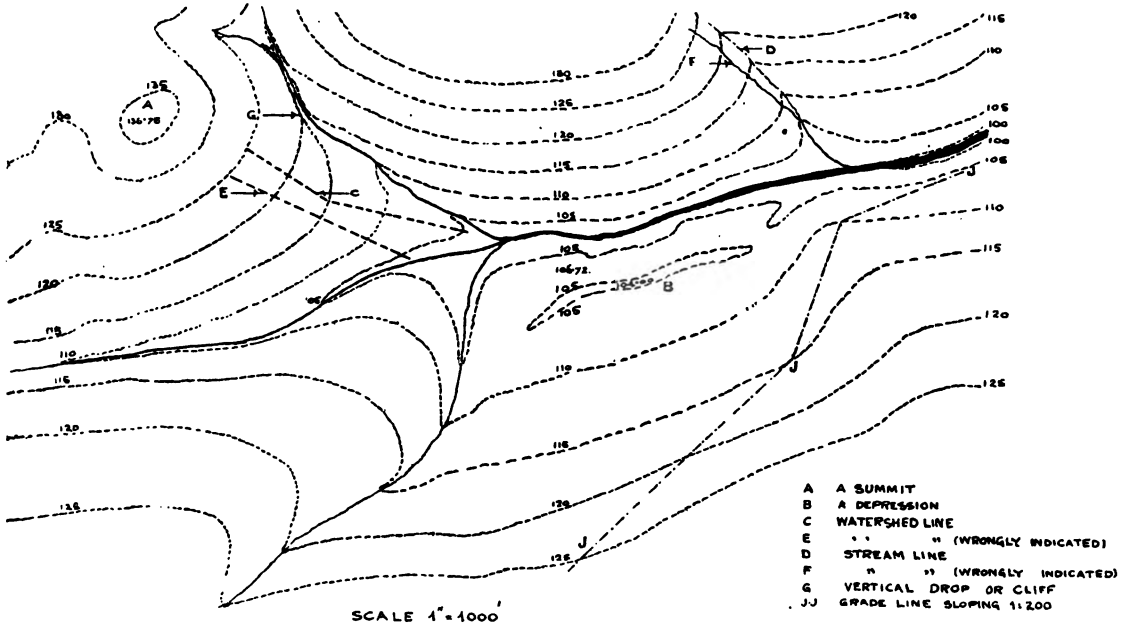


FIG. 22.—Characteristics of Contours.

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(see G) where, being vertically superimposed, they coincide in plan, and can only intersect one another in cases such as overhanging cliffs or river banks. Any line, rising or falling at a uniform rate, or gradient, will cross successive contour lines at uniform intervals of its length. To lay out on the map such a line, which may be the approximate trace of a railway or canal, it is therefore only necessary to take in the dividers the length which corresponds, on the desired gradient, to the contour interval of the plan; step this length off from contour to contour, and draw a line through the points thus marked. See line JJJ, which coincides with a grade line of 1 in 200 at each point where it touches a contour.

### Field Sketches in Drawing Contours.

Small sketches made in the "Remarks" column of the field book, will more than repay, in office work, the extra expenditure of time, at points where the topography is difficult to record by means of measurements alone.

### Method of Interpolating Contours from Spot-Levels.

A short examination of Plate I will show the method of drawing in, or rather sketching in, contour lines. They are at best but a close approximation to truth, for to make them very accurate would make the work unreasonably costly by entailing the observation of an enormous number of points in the field.

The contour interval on Plate I is 5 feet. Station A is at R.L. 1000·00; this is evidently one point on the 1000·00 contour. South of A, and between A and B, are two spot-levels, R.L. 1001·00 and 994·55, at the river bank. The 1000-foot contour must pass between these and close to the former. The distance between the two points scales 125 feet, and their difference of level is 1001·00 − 994·55 = 6·45 feet. There-

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fore the distance from spot-levels 1001·00 to the R.L. 1000·00 contour is $\frac{1}{6{\cdot}45}$ of 125 feet = 19 feet. (It is useless to work out these distances to decimals, as they can only be scaled off to the nearest 2 feet or so.) The distance thus found is laid off, and the contour is drawn to it from A. The level of B is 1005·05, while that of the river bank opposite to B is, by interpolation from the two nearest spot-levels upon it, R.L. 995·22 at a point halfway between the latter. In the distance of 240 feet from B to this point there is a difference of level of 1005·05 − 995·22 = 9·83 feet. The 1000-foot contour, therefore, passes $\frac{5{\cdot}05}{9{\cdot}83} \times 240$ feet = 129 feet approximately from B towards the point 995·22 on the river bank, and is so drawn. In a similar manner its position is fixed in relation to that of the other spot-levels, and the line representing it is drawn through the points thus found. The other contours are fixed in the same way. The proportion sums are either worked out mentally or performed on a slide rule, which is less troublesome; but for rough work the position of the contours can often be estimated by eye with sufficient accuracy.

## Radial Scale for making Interpolations.

The device shown in fig. 23 can be used for interpolating other heights both between spot-levels and between contours already drawn; its use avoids all calculation and saves much time. The radiating lines are drawn on a small piece of tracing-cloth and numbered as illustrated; a slip of wood which can work against the edge of a rolling parallel ruler is fastened to one end of the piece of tracing-cloth. Suppose the position of R.L. 990 contour, between two spot-levels whose values are respectively 987·50 and 992·50, to be required. A parallel ruler is placed so that it lies parallel to a line joining

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these two spot-levels on the plan, and the wooden slip A is placed against its edge in such a position that the lower spot-level, 987·50, is seen through the tracing-cloth to lie midway between the radial lines numbered 7 and 8 on the

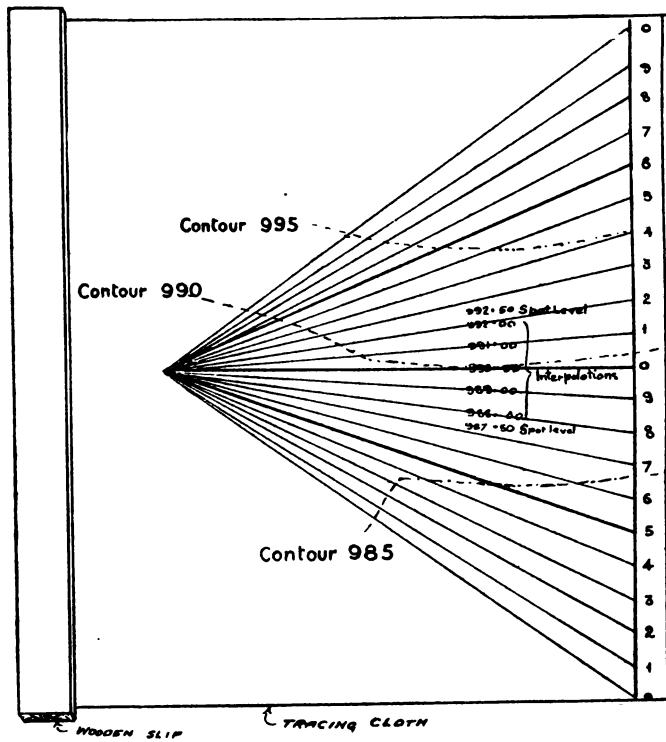


FIG. 23.—Interpolator Scale for use in plotting Contours.

lower half of the interpolator scale. By advancing or retiring the parallel, and slightly moving slip A against its edge, in order that the 987·50 point may remain always between lines 7 and 8, the interpolator is slipped along over the drawing until the other figure, 992·50, is seen through the

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tracing-cloth to be just midway between radial lines 2 and 3 on the upper half of the scale.

The radial lines marked 8, 9, 0, 1, 2, now indicate the positions on the paper between the two given spot-levels for contours at R.L.'s 988·00, 989·00, 990·00, 991·00, and 992·00, which can be pricked through. It will be seen that the figure marked against each radial line, when it is thus brought into position, corresponds with the figure in the units place of the reduced level which the line indicates. The interpolator can be used without the parallel ruler, which is merely a convenience in keeping the base line AB parallel with a line joining the points between which interpolations are desired.

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## CHAPTER VII.

### *CALCULATION OF LINES AND AREAS FROM CO-ORDINATES.*

If it be required to extend the survey shown on Plate I in the absence of the original field notes, while the only measurements to which reference can be made are the co-ordinates written on the plan, and the only two pegs which can be found in the field are those at B and H, not directly connected by a line in the original survey, the following course can be adopted. In order to recover the exact meridian of the original survey, so that it can be used for the extension and for the location of other original points, it is necessary to calculate the *bearing* of the line BH from the co-ordinates. These are—

| | Cos. | Sin. |
|---|---|---|
| For B . . . | −565·838 | −102·235 (p. 88) |
| For H . . . | −397·596 | +760·735 (p. 91) |
| and their differences are | 168·242 | 862·970 |

Station H, having the smaller minus value in latitude, is 168·24 feet North of B. In departure, B is −102·23 feet, or this distance *West* of the zero line, and H is 760·73 feet East of the same line, so that altogether H is 862·97 feet East of B. Thus the points B and H lie as sketched below (fig. 24).

The angle $\theta$ is the reduced bearing of the line BH. But



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from the sketch it is evident that BH passes between 0 and 90° on the circle, so that the reduced bearing and the bearing are identical. (If BH fell between 90° and 180°, its reduced bearing deducted from 180° would give the bearing of the line; between 180° and 270° it would be necessary to add 180°, and between 270° and 360° to subtract from 360° (see p. 82), since the reduced bearing is the smallest angle between the line and the meridian.

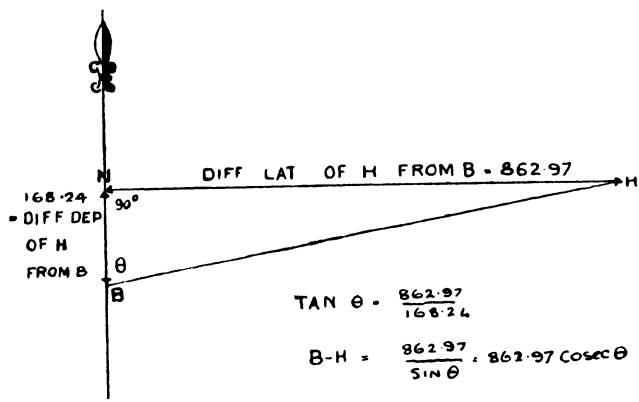


FIG. 24.—Diagram illustrating Calculation of Bearings and Distances from Co-ordinates.

In the right-angled triangle BNH, the ratio $\frac{\text{Perpendicular}}{\text{Base}}$, or $\frac{862{\cdot}97}{168{\cdot}24}$, is the tangent of the angle $\theta$, and from a table of tangents it is found that this ratio $\frac{862{\cdot}97}{168{\cdot}24}$, or 5·1294, is the tangent of 78° 58′, which is therefore the reduced bearing from B to H, and, in this case, corresponds with the bearing. The length BH can be obtained from "Length of line" = $\sqrt{(\text{diff. lat.})^2 + (\text{diff. dep.})^2}$, or, since the difference of departure

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862·97 is BH sin $\theta$, the value of BH is $\frac{862{\cdot}97}{\sin\theta} = 862{\cdot}97 \times \text{cosec}\,\theta = 879{\cdot}22$ feet. If the traverse be not a closed one, the bearing and distance between the end stations can be calculated in order to check the plotting.

### Supplying Missing Data.

The same method can be used to calculate the bearing and length of a side omitted in the field notes, and since, in any closed traverse :—

$$L_1 \cos B_1 + L_2 \cos B_2 + L_3 \cos B_3 + \text{etc.} = 0.$$
$$L_1 \sin B_1 + L_2 \sin B_2 + L_3 \sin B_3 + \text{etc.} = 0,$$

where $L_1$, $L_2$, etc., are the lengths of the successive sides and $B_1$, $B_2$, etc., their bearings (that is, the algebraic sum of all the differences of latitude is zero, as is also that of the differences of departure), these two simultaneous equations can be solved for any two items missing in the data, *e.g.* the lengths of two sides, or the length of one side and the bearing of another.

### Use of Co-ordinates in Closure of Traverse by Observation to Distant Reference Points.

By using the foregoing principles, a long traverse can frequently be closed which would otherwise remain unchecked. Let A, B, C, D . . . G be successive stations of such a traverse (fig. 25).

A church spire S or other conspicuous object is visible from stations A, D and G, and bearings are read to it from these points. From the co-ordinates of the traverse the bearing and length of the straight line A–D are calculated, the triangle ADS can then be solved and co-ordinates of S obtained, since bearings AS and DS were observed in the field. Similarly, the bearing and length of DG are calculated, the triangle DGS is solved, and other co-ordinate values for S are obtained which should agree with the values first found.

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If no reference object, such as S, is visible from both ends of the whole traverse, it may be possible to observe one such point from, say, stations, A, C, and E, and a different one from E, F, and G, thus checking the work in two independent portions; or if such an object is visible from both ends of the traverse only, and bearings are read to it from thence, the angular work can be checked by the precept

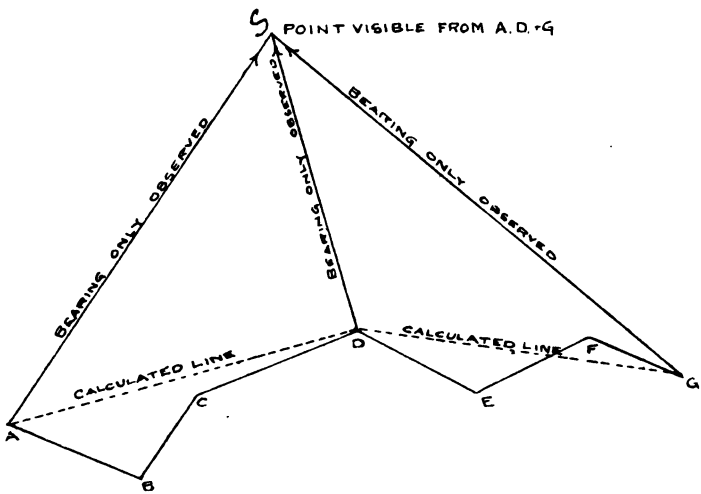


FIG. 25.—Check Bearings to Distant Object.

"Sum of interior angles = twice as many right angles as figure has sides, less four right angles."

## Calculation of Areas from Co-ordinates.

In the following demonstration of the principles involved, and the practical office methods used in the calculation of areas from co-ordinates, the area of the figure ABCDEFGH (Plate I) is worked out in three different ways. The co-ordinates themselves of the points A, B, C, etc., were worked out on pp. 88 to 91.

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## Modification of Co-ordinate Values for Calculation of Areas.

It will be more convenient for purposes of calculation to have the latitudes and departures of the points either all upon one side or all upon the other of their respective zero lines, in order that the + and — signs indicative of direction

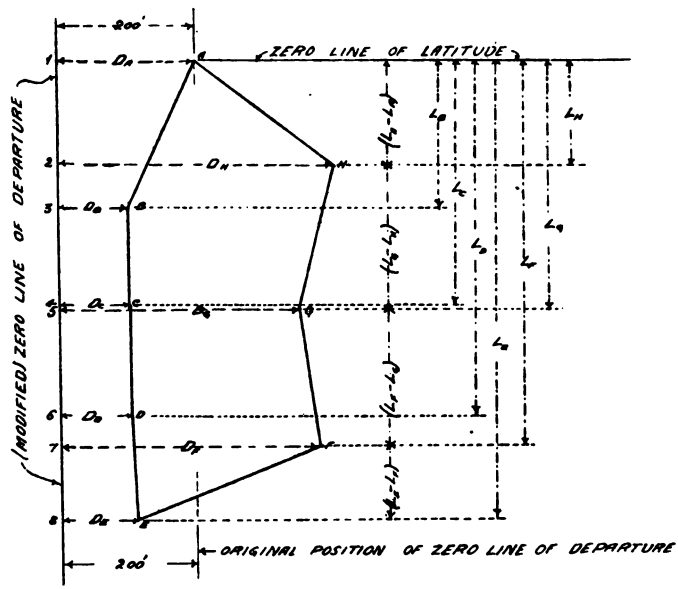


FIG. 26.—The Calculation of Areas from Co-ordinates.

may be disregarded. The greatest minus value in departure is at station D (= 110·71). By adding +200 to each departure a positive value for all stations will be obtained (see fig. 26), and this of course will not change the *relative* position of the points themselves in any way. The latitudes already have the minus sign throughout, the points all falling South of the zero line of latitude; consequently, there is no need to change them.

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The reversed numerical values thus obtained for the co-ordinates are (see pp. 88–91) :—

| | Latitudes. | Departures. | |
|---|---|---|---|
| A | 0 | 200 | (instead of ± 0) |
| B | 565·84 | 97·76 | |
| C | 979·83 | 99·27 | |
| D | 1469·73 | 89·29 | |
| E | 1899·70 | 94·86 | |
| F | 1595·51 | 912·08 | |
| G | 1001·75 | 812·81 | |
| H | 397·60 | 960·74 | |

## General Principles.

In fig. 26 the required area ABCDEFGH is the area of the figure 1AHGFE8 less that of the figure 1ABCDE8. Each of these figures is composed entirely of trapezoids as 1AH2, 3BC4, etc., and the latitudes and departures $L_A$, $L_B$, $D_A$, $D_B$, etc., of the various points A, B, C, ... give the dimensions of these trapezoids. Area ABCDEFGH = (Area of trapezoids 1AH2, 2HG5, 5GF7, and 7FE8) minus (area of trapezoids 1AB3, 3BC4, 4CD6, and 6DE8). Since the area of a trapezoid is (mean of two parallel sides × vertical distance between them) the following formula is obtained :—

Required area ABCDEFGH ;

$$\left\{\left(L_H - L_A \times \frac{D_H + D_A}{2}\right) + \left(L_G - L_H \times \frac{D_G + D_H}{2}\right) + \left(L_F - L_G \times \frac{D_F + D_G}{2}\right) + \left(L_E - L_F \times \frac{D_E + D_F}{2}\right)\right\}$$
$$- \left\{\left(L_B - L_A \times \frac{D_B + D_A}{2}\right) + \left(L_C - L_B \times \frac{D_C + D_B}{2}\right) + \left(L_D - L_C \times \frac{D_D + D_C}{2}\right) + \left(L_E - L_D \times \frac{D_E + D_D}{2}\right)\right\} \quad . \quad (1)$$

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Putting the common denominator, 2, on the other side of the equation :—

$$\begin{aligned} 2 \times \text{Area} = [&\{(L_H - L_A)(D_H + D_A) + (L_G - L_H)(D_G + D_H) \\ &+ (L_F - L_G)(D_F + D_G) + (L_E - L_F)(D_E + D_F)\} \\ &- \{(L_B - L_A)(D_B + D_A) + (L_C - L_B)(D_C + D_B) \\ &+ (L_D - L_C)(D_D + D_C) + (L_E - L_D)(D_E + D_D)\}]. \end{aligned}$$

Multiplying out, removing brackets, and cancelling :—

$$\begin{aligned} 2 \times \text{Area} = [&D_A L_H - D_H L_A + D_A L_H - D_A L_A + D_G L_G - D_G L_H \\ &+ D_H L_G - D_H L_H + D_F L_F - D_F L_G + D_G L_F \\ &- D_G L_G + D_E L_E - D_E L_F + D_F L_E - D_F L_F \\ &- D_E L_B + D_B L_A - D_A L_B + D_A L_A - D_C L_C \\ &+ D_C L_B - D_B L_C + D_B L_B - D_B L_D + D_D L_C \\ &- D_C L_D + D_C L_C - D_E L_E + D_E L_D - D_D L_E \\ &+ D_E L_D] \end{aligned}$$

and re-arranging the figures :—

$$\begin{aligned} 2 \times \text{Area} = &D_B L_A - D_A L_B + D_C L_B - D_B L_C + D_D L_C - D_C L_D \\ &+ D_E L_D - D_D L_E + D_F L_E - D_E L_F + D_G L_F \\ &- D_F L_G + D_H L_G - D_G L_H + D_A L_H - D_H L_A \end{aligned} \quad (2)$$

Even thus simplified, the equation 2 above appears formidable, but the following simple arrangement * of its symbols removes any difficulty or confusion :—

$$2 \times \text{Area} = \left(\frac{D_A}{L_A} \times \frac{D_B}{L_B} \times \frac{D_C}{L_C} \times \frac{D_D}{L_D} \times \frac{D_E}{L_E} \times \frac{D_F}{L_F} \times \frac{D_G}{L_G} \times \frac{D_H}{L_H} \times \frac{D_A}{L_A}\right) \quad (3)$$

In 3 it follows from equation 2 that the difference between (Sum of products of figures joined by full lines) and (Sum of products of figures joined by dotted lines) is equal to twice the area sought.

* Breed and Hosmer, *Principles and Practice of Surveying* (Chapman & Hall, Limited).

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## Office Methods.

### First Method.

Briefly, therefore, the whole process of thus calculating the area of an enclosure from the co-ordinates of its corners, is as follows :—

(1) Make the $\left\{\begin{matrix}\text{latitudes}\\\text{departures}\end{matrix}\right\}$ either all positive or all negative by adding a constant positive or negative round figure to each $\left\{\begin{matrix}\text{latitude}\\\text{departure}\end{matrix}\right\}$. If possible, choose this constant in such a way that the new values will be numerically less than the original ones, in order to reduce the subsequent labour of multiplication. The constant added to the latitudes need not be the same as that added to the departures, either in sign or amount.

(2) Disregarding their signs, arrange the new co-ordinates thus found, as in fractional-form equation 3 above, taking the points on the perimeter of the enclosure in consecutive order, until the starting-point is again reached, and join up the numerators and denominators with full and dotted lines as shown in (3). Multiply together each pair of figures joined by a full line, and total the products; do the same with regard to the pairs of figures joined by dotted lines. The area is half the difference between the two totals thus obtained. This calculation can be arranged in tabular form, thus :—

*Example of Calculation by First Method.*

For the figure ABC . . . H (Plate I).

$2 \times \text{Area} =$

| | | | | |
|---|---|---|---|---|
| 0 | 565·84 | 979·83 | 1469·73 | 1899·70 |
| 200 | 97·76 | 99·27 | 89·29 | 94·86 |

| | | | |
|---|---|---|---|
| F. | G. | H. | A. |
| 1595·51 | 1001·75 | 397·60 | 0 |
| 912·08 | 812·81 | 960·74 | 200 |

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| Co-ordinates. | Logarithms. | Sums of Logs. | Numbers to Sums of Logs. | |
|---|---|---|---|---|
| | | | − | + |
| 565·84 × − 200·00 | 2·3010300 | 5·0537236 | 113,168·00 | |
| 565·84 × − 200·00 | 2·7526936 | | | |
| 565·84 × + 99·27 | 1·9968180 | 4·7495116 | | 56,170·92 |
| 979·83 × − 97·76 | 1·9901612 | 4·9813119 | 95,788·18 | |
| 979·83 × − 97·76 | 2·9911507 | | | |
| 979·83 × + 89·29 | 1·9508028 | 4·9419535 | | 87,489·00 |
| 1469·73 × − 99·27 | 1·9968180 | 5·1640556 | 145,900·10 | |
| 1469·73 × − 99·27 | 3·1672376 | | | |
| 1469·73 × + 94·86 | 1·9770831 | 5·1443207 | | 139,418·59 |
| 1899·70 × − 89·29 | 1·9508028 | 5·2294878 | 169,624·18 | |
| 1899·70 × − 89·29 | 3·2786850 | | | |
| 1899·70 × +912·08 | 2·9600329 | 6·2387179 | | 1,732,678·09 |
| 1595·51 × − 94·86 | 1·9770831 | 5·1799826 | 151,350·07 | |
| 1595·51 × − 94·86 | 3·2028995 | | | |
| 1595·51 × +812·81 | 2·9099890 | 6·1128885 | | 1,296,846·27 |
| 1001·75 × − 912·08 | 2·9600329 | 5·9607923 | 913,676·17 | |
| 1001·75 × − 912·08 | 3·0007594 | | | |
| 1001·75 × +960 74 | 2·9826059 | 5·9833653 | | 962,421·52 |
| 397·60 × − 812·81 | 2·9099890 | 5·5094354 | 323,173·26 | |
| 397·60 × − 812·81 | 2·5994464 | | | |
| 397·60 × +200 | 2·3010300 | 4·9004764 | | 79,520·00 |
| | Totals . . | | 1,912,679·96 | 4,354,544·39 |
| | | | | 1,912,679·96 |
| | Difference = Double area = | | | 2,441,864·43 |
| | Area, sq. ft. = | | | 1,220,932·22 |

## Second Method.

The above method, although very simple, involves the multiplication of somewhat large figures. In this respect some advantage is gained by writing equation (2) in the form

$$\begin{aligned}\text{Double area} = {} & L_A(D_B - D_H) + L_B(D_C - D_A) + L_C(D_D - D_B) \\ & + L_D(D_E - D_C) + L_E(D_F - D_D) + L_F(D_G - D_E) \\ & + L_G(D_H - D_F) + L_H(D_A - D_G)\end{aligned}$$

or, thus—

$$\begin{aligned}\text{Double area} = {} & D_A(L_H - L_B) + D_B(L_A - L_C) + D_C(L_B - L_D) + \text{etc.} \\ & + D_H(L_G - L_A)\end{aligned}$$

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that is—

Multiply the $\frac{\text{lat.}}{\text{dep.}}$ close of each point by $\frac{\text{dep.}}{\text{lat.}}$ of point next preceding it, *minus* $\frac{\text{dep.}}{\text{lat.}}$ of that next following it; the sum of these products is twice the area sought.

This calculation for the area of AB . . . H is as follows, the result being the same as before :—

*Example of Calculation by Second Method.*

| Latitudes. | Departures. | Difference of Adjacent Latitudes. | Logs. | Sums. | Double Area. − | Double Area. + |
|---|---|---|---|---|---|---|
| H. 397·60 | ... | ... | ... | ... | ... | ... |
| A. 00 | 200·00 | − 168·24* | 2·3010300<br>2·2259293 | 4·5269593 | 33,648·00 | ... |
| B. 565·84 | 97·76 | − 979·83 | 1·9901612<br>2·9911507 | 4·9813119 | 95,788·18 | ... |
| C. 979·83 | 99·27 | − 903·89 | 1·9968180<br>2·9561156 | 4·9529336 | 89,729·17 | ... |
| D. 1469·73 | 89·29 | − 919·87 | 1·9508028<br>2·9637265 | 4·9145293 | 82,135·19 | ... |
| E. 1899·70 | 94·86 | − 125·78 | 1·9770831<br>2·0996116 | 4·0766947 | 11,931·49 | ... |
| F. 1595·51 | 912·08 | + 897·95 | 2·9600329<br>2·9532522 | 5·9132851 | ... | 819,002·26 |
| G. 1001·75 | 812·81 | +1197·91 | 2·9099890<br>3·0784242 | 5·9884132 | ... | 973,673·18 |
| H. 397·60 | 960·74 | +1001·75 | 2·9826059<br>3·0007594 | 5·9833653 | ... | 962,421·52 |
| A. 00 | ... | ... | ... | ... | ... | ... |
| | | | Totals . . | | 313,232·03 | 2,755,096·96<br>313,232·03 |
| | | | Difference = Double area = | | | 2,441,864·93 |
| | | | Area, sq. ft. = | | | 1,220,932·46 |

* 397·60 − 565·84 = − 168·24, paying due regard to signs.

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### Third Method.

A third method is to apply equation (1) (p. 107) as it stands, thus :—

Add to the departure of each point that of the point next following, and subtract from the latitude of each point that of the one next following, continuing in the same direction around the enclosure until the starting-point is again reached. The sum of the products of the pairs of figures thus obtained is twice the area. Putting this calculation into tabular form for the area AB . . . H (Plate I), as before, a similar result follows :—

*Example of Calculation by Third Method.*

| Dep. | Lat. | Ft. | Ft. | Product. Sq. ft. | Product. Sq. ft. |
|---|---|---|---|---|---|
| | | | | − | + |
| A + B × | A − B = | 297·76 × | − 565·84 = | 168,484·52 | |
| B + C × | B − C = | 197·03 × | − 413·99 = | 81,568·45 | |
| C + D × | C − D = | 188·56 × | − 489·90 = | 92,375·54 | |
| D + E × | D − E = | 184·15 × | − 429·97 = | 79,178·97 | |
| E + F × | E − F = | 1006 94 × | + 304·19 = | ... | 306,301·08 |
| F + G × | F − G = | 1724·89 × | + 593·76 = | ... | 1,024,170·69 |
| G + H × | G − H = | 1773·55 × | + 604·15 = | ... | 1,071,490·23 |
| H + A × | H − A = | 1160·74 × | + 397·60 = | ... | 461,510·22 |
| | | | Totals . . | 421,607·48 | 2,863,472·22 |
| | | | | | 421,607·48 |
| | | | Difference = | Double area = | 2,441,864·74 |
| | | | | Area, sq. ft. = | 1,220,932·37 |

It will be noticed that, proceeding round the figure in the *reverse* direction, viz. from A through H, G, F, and so on back to A, the minus products would have been the greater, and the resulting double area would have had the same numerical value with the minus sign. But this would be of no importance, as it is only the numerical value of the area which is wanted.

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## The Planimeter.

The most practical method of obtaining the area of an irregular figure, such as a field bounded by a stream and other crooked or curved boundaries, is to measure the plan itself with a planimeter. Fig. 27 shows an ordinary pattern of this instrument.

In principle it is an integrating machine, and consists of an arm S of variable length sliding in a carriage C to which is attached an arm of fixed length A. On the carriage are an index disc G, reading in units, a graduated wheel or drum W,

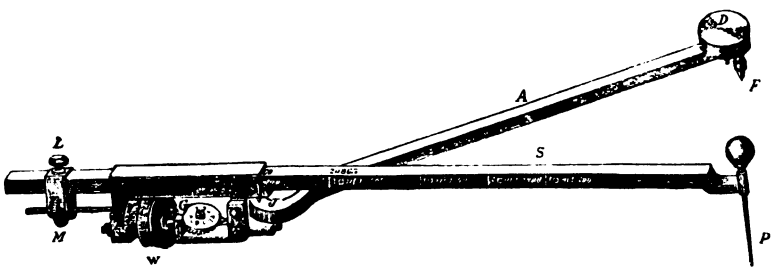


FIG. 27.—Planimeter.

reading 10ths and 100ths, and a vernier V reading 1000ths into the scale on W. The index mark J can be set to any desired graduation on S by means of the fine adjustment M and lock-screw L.

In using the instrument, the fixed point F is pressed lightly into the paper, the weight D being sufficient to keep it there, usually *outside* the boundary lines on the plan, and the tracing-point P is passed around those lines in the direction of clock-hands and returned to the starting-point. The difference between the readings of the scales G, W, and V before and after tracing out the lines with the point P, indicates the area enclosed by those lines.

The graduations on the side of bar S are arranged by the maker to suit scales in common use; others can be determined



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by trial and error. The dimension engraved against each graduation mark on the side of S, on English instruments, is the value of a division on disc G, or of one revolution of drum W, when the planimeter is used with the index J set to that mark. The figures on the *top* of bar S against each mark are only used when the fixed point is placed *within* the area measured, in which case the figure against the index mark is to be *added* to the difference of the readings. Foreign instruments are sometimes marked only for metric units on the bar S, and have the *vernier* unit ($\frac{1}{1000}$ of a division on G) engraved against the marks. It is usually convenient to set J either to the mark for square centimetres or to that for square inches, and to convert the resulting measurements in square centimetres or square inches *on the plan* into the desired unit. One square inch of paper on a plan to a scale of 600 feet = 1 inch represents of course 360,000 square feet of area on the ground.

### Points to be observed in Using Planimeter.

The following points should be observed in using a planimeter :—

Select a position for the fixed point F so that, in the travel of the tracing-point round the figure to be measured, the two arms A and S do not approach each other very closely; otherwise the instrument is liable to topple over. If the position selected is *outside* the area, there is less trouble in calculating results. Do not attempt to set the scales to zero, but note their readings before and after completing the circuit with the tracing-point, and take the difference; do not rely upon one measurement only of the area, but take the mean of two or more determinations. Avoid touching the steel rim of the wheel W with the fingers; it is easily rusted, and any consequent irregularity in its rolling and sliding motion will affect the accuracy of the instrument.

In measuring areas from plans, do not rely implicitly upon

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the graduations of the bar S. Measure with the instrument a test square or circle drawn on the plan itself, and thus obtain a percentage correction to be applied to the results as given by the planimeter scales. If no scale appears on the plan, the length of a line as plotted can be used as the side of a test square (the true length of this line being known from the field notes), or one of the co-ordinate squares can be used.

Guide the tracer along straight lines with a set-square or other straight-edge.

If the tracing-point is, while measuring, accidently moved slightly off a line to one side, equalise matters as far as possible by subsequently bringing it over to the other side to an equal extent and for a similar distance. Hold the tracing-point very lightly, and do not scratch the paper with it.

Enter all readings on a proper sheet, in an orderly manner, and with a reference to the plan concerned.

### Accuracy of Planimeter Measurement.

There is no better means of computing irregular areas than is afforded by a good planimeter, but absolute accuracy must not be expected. Irregular lines will not be traced with the pointer so closely as those of a test square or circle, and all errors of draughtsmanship will affect the results obtained.

A straight line cannot be plotted with a smaller lateral deviation than of about $\frac{1}{200}$th of an inch from its mathematical position, nor is there any certainty of following lines with the tracing-point to this degree of accuracy. Allowing for a total deviation of $\frac{1}{200}$th inch from the truth on a plan drawn to a scale of 600 feet = 1 inch, this error would amount to $\pm 0{\cdot}005$ square inch or $360{,}000 \times 0{\cdot}005 = 1800$ square feet of area per inch of line, if the error were continually made in the same direction. But the number of errors liable to be made in each direction being equal, they tend to neutralise

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one another, and thus leave only a small residual error in the final result.

The percentage of residual error is of course variable, and, apart from the personal equation, depends to some extent on the shape of the figure measured. Theoretically, it should be least on a circle and greatest on a long and very narrow strip. Experiment confirms this, and the average percentage seems to be in the neighbourhood of 0·05 per cent. on circles and squares, 0·75 per cent. on fairly narrow rectangles, and a higher rate on many-sided rectilinear figures, in the case of operators having average skill. It also appears that increasing the number of angles which entail sudden changes of direction in the path of the tracing-point, decreases the accuracy of the result to some extent. Probably this is merely due to slight irregularities of manipulation in making such changes of direction. But, in the case of figures bounded by irregular lines, the results obtained probably exceed in accuracy the precision with which such boundaries are delineated on the plan.

## Example of Planimeter Measurement, and Comparison with Calculated Area.

The following planimetric measurement of the figure AB . . . H (Plate I), compared with the calculated area 1,220,932 square feet, will give a fair idea of the results which may be expected.

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## Area by Planimeter of Figure AB . . . H.
(See Plate I.)

| Readings of Planimeter. | | | Mean. Sq. cms. | Remarks. |
|---|---|---|---|---|
| 1st Reading. | 2nd Reading. | 3rd Reading. | | |
| Test Square. | (Co-ordinate | square 500′ | × 500′) | Planimeter unit, = 100 sq. cm. |
| 2·086 | 2·849 | 3·247 | | |
| 2·489 | 3·252 | 3·650 | | Scale of plan 1″ = 200′. |
| Diff. = 0·403 | 0·403 | 0·403 | 40·3 | 1 sq. cm. = ·155 × 40,000 = 6200 sq. ft. |

40·3 sq. cms. = 249,860 sq. ft.
True area = 250,000 ,,

Error = 140 ,, Result is too small by, say, 0·06 per cent.

| | | | | |
|---|---|---|---|---|
| Figure ABC . . . H. | | | | |
| 1·364 | 3·330 | 5·298 | | |
| 3·330 | 5·298 | 7·262 | | |
| 1·966 | 1·968 | 1·964 | 196·66 | |

196·66 sq. cm. = 1,219,292 sq. ft.
Add 0·06 per cent. 732 ,,

Total 1,220,024 = Area ABC . . . H as determined by planimeter.
1,220,932 True area as calculated.

Error 908 sq. ft. Result is too small by, say, 0·07 per cent.

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## CHAPTER VIII.

### *ADJUSTMENTS OF THE TACHEOMETER.*

THE adjustments of the tacheometer, commonly called its "permanent" adjustments (although such a description really only applies to those which rest with the makers and cannot be altered, such as making the vertical axes truly concentric, etc.), are those of

1. The level bubbles on the horizontal plate.
2. The collimation.
3. The telescope and vernier-arm bubbles.
4. The zeros of the arc verniers.
5. The standards.
6. The anallatic lens.

Before adjustments can be made, care must be taken that the tripod is very firmly set up, and that there are no loose screws or shaky connections in the tripod itself.

### The Plate Bubbles.

These must be so adjusted that the horizontal plate is truly horizontal when they are at the centre of their runs.

1*st Method.*—Set the index of one vernier (say the A vernier) on the horizontal plate to 360°, turn the instrument on the lower axis until the longer level tube lies parallel with two of the footscrews, clamp the lower axis, release the upper plate, and bring the level bubble to the centre of its run. Now, without clamping the plates, make the same vernier read



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90° as near as can be judged by eye; this places the level bubble parallel to the line of the third footscrew (or the other pair if there be four). Bring the bubble to the centre in this position. Turn back to 360°, and again correct, if necessary. Next, bring the *other* vernier ( B ) to 360°, thus reversing the level tube end for end. If the bubble now leaves the centre, correct the error, half by one footscrew and half by the capstan-headed nuts at the end of the bubble tube. Do not force these nuts: a very moderate pressure suffices.

Turn the vernier B to 90° and correct with the third footscrew only; then turn back B again to 360° to check, and correct, if necessary, with one footscrew only of the first pair. The bubble should now remain in the centre of its run while the instrument is turned through a complete revolution. It probably will do so nearly, in which case the operation must be repeated for exactitude.

*2nd Method.*—Set up the instrument with the A vernier at 360° and place the *telescope* bubble parallel to two footscrews, having the vertical circle vernier set to zero. Level up exactly as before with reference to this bubble, but after reversing it by bringing the B vernier to 360°, correct half by the footscrew and half by the *vertical-arc tangent screw.* The bubble, although a very sensitive one, can thus easily be made to " reverse " exactly. When it does so, the vertical axis is truly vertical, and the horizontal plates are truly horizontal; the *whole* error in the bubbles on them can therefore be corrected at once by their capstan-headed screws alone.

## The Collimation Lines, Vernier Zeros, and Vernier Arm Bubble.

(*a*) The line of sight through the central horizontal wire must now be made truly horizontal when the telescope bubble is at the centre of its run, and at the same time the vertical arc verniers must read zero.

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If method 2 was adopted for the plate bubbles, the instrument will be standing with the telescope bubble centred and reversing perfectly. If not, make it so, by means of the vertical-arc tangent screw and the footscrews, as described, and while it is exactly centred note the reading of the central horizontal wire on a level staff, firmly fixed as far away as it can be read with precision. Then set the vertical-arc vernier exactly to zero, and with the clipping screws or screw E (fig. 5) bring back the centre wire to the same reading on the staff without touching the tangent screw. The telescope bubble will then be at the centre of its run, when the vertical-circle verniers are at zero. The vernier-arm bubble can now be corrected by means of its capstan-nuts alone, to agree with the telescope bubble.

### The Horizontal Web.

(*b*) The next procedure is to make the line of sight truly horizontal. Revolve the telescope in its bearings, so that the bubble is underneath, set the vertical-arc verniers to zero, and again read the staff. If the reading is the same as that first obtained when the bubble was on top and the verniers were at zero, then the adjustment is correct. If not, the true horizontal line cuts the staff midway between the two readings observed. Make the central wire intersect the mean of the two staff readings by the vertical pair of diaphragm screws BB (fig. 5). Check again with the bubble uppermost and in the centre of its run, and remove any residual error.

The first part (*a*) of this adjustment has to be examined, and probably re-made, every time the instrument is taken from its box, unless it is arranged for packing in one piece. The second part (*b*) is so important that it must be frequently checked.

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## The Vertical Web.

The line of sight through the vertical wire, if there be one, or through the intersection of the diagonal cross wires where such are used, must now be adjusted to be perpendicular to the transit axis of the telescope. If it be not perpendicular, the instrument will not "throw over" a line correctly—that is, when the instrument is placed in the middle of a straight line, the line of sight will not fall upon its extremities when the telescope is revolved.

The instrument being carefully levelled up, turn back the covers of the telescope bearings and direct the central intersection of the cross wires to some well-defined distant point. Gently loosen one of the clipscrews E (fig. 5), and, without disturbing the lower part of the instrument, lift the telescope out and reverse the transit axis in its bearings. Release the clamp to the vertical circle and transit the telescope, directing it again to the test object. If the latter falls exactly on the cross hairs as before, the adjustment is correct; but if not, correct *half* the error by means of the horizontal pair of diaphragm screws BB, and *half* by the horizontal plate tangent screw. Replace the telescope axis in its original position to check, and repeat until perfect.

## The Standards.

In order that the transit axis may be perpendicular to the vertical axis of the instrument, the standards carrying the telescope must be of precisely equal height. In many instruments there is no means of adjustment provided by which their height can be altered after it is once fixed by the maker. In others there is a slit in one standard which can be slightly opened or closed by opposing screws, thus lowering or raising one end of the transit axis; while in a third type, a vertically adjustable block is placed under one bearing. This pattern is more liable to derangement than the second, which can only be displaced by exceedingly rough usage, and is capable

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of very fine adjustment. Probably, however, the first-mentioned type is really the best, as it can only be deranged by long use causing wear on the bearings, which an instrument maker can correct without difficulty.

The adjustment can be effected either with a striding level (which is a movable level fitted with bearings to stand on the ends of the transit axis) or in the manner described later.

### Adjustment of Standards by Striding Level.

To check the adjustment, level up the instrument exactly, by means of the telescope bubble (which is the most delicate one), using the vertical circle tangent screw and the foot-screws, until this bubble remains central in any position. The vertical axis is then truly vertical. Having done this, turn back the caps covering the ends of the telescope axis, and erect the striding level upon the latter. Observe and note down carefully the readings of the ends of its bubble upon the scale marked on the tube; put a chalk mark on the high end if the bubble is not in the centre. Turn the upper limb through 180°; if the adjustment of the standards is correct, the bubble will stand in exactly the same position as before, *i.e.* the chalk-marked end will still be the high end, and the bubble readings will be unchanged. If not, turn the adjusting screw or screws to raise or lower one end of the axis, correcting half the error each time, until it is entirely eliminated. By this method of reading the ends of the bubble, in whatever position it may be, the troublesome work of adjusting the striding level itself is avoided.

### Adjustment of Standards without Striding Level.

The same adjustment can be effected without a striding level in the following manner:—Set up the instrument at about forty or fifty feet distance from a high wall or other object of considerable elevation, level it exactly, and direct the

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telescope to some small point situated as high up as possible, and bring the cross lines to bear exactly upon it. All parallax must now be carefully eliminated by means of the focussing screw and by gently moving the eyepiece in or out of its socket until the cross lines and the object are brought exactly into the same focus. Then tilt the telescope and make a small mark on the wall near to the foot, at the point of intersection of the cross lines. Now turn the upper plate through 180°, transit the telescope, and again direct it to the upper mark, after which tilt it as before and see if the point of intersection of the cross lines exactly corresponds with the lower mark. If it does so, the transit axis is truly horizontal; but if not, half the error must be corrected by means of the adjusting screws as previously described, and the operation repeated in order to ensure accuracy.

### The Anallatic Lens.

This, in instruments providing means for its adjustment, is moved by a key inserted at the side of the telescope. Measure out with a steel band or chain accurately a distance of, say, 300 feet from the centre of the instrument. With the telescope horizontal, take the readings of the stadia wires on the staff. If these readings do not agree exactly with the measured distances, slightly turn the key of the lens until correct readings are obtained.

### Common Faults of Construction.

If the level bubbles have been adjusted with the vertical circle end of the telescope axis over the A vernier, the instrument should be preferably set up in the same way for subsequent use.

### Lack of Rigidity and Balance.

It will not infrequently be found that owing to lack of rigidity and to want of correct counterbalancing for the

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weight of the vertical circle and clamps, etc., the adjustment of the level bubbles is affected by reversing the telescope axis end for end in its bearings. Faults of this kind are usually due to reckless economy in the weight of instruments.

### Unsteadiness and Vibration in a Wind.

If the amount of metal in a tacheometer, and of wood in its tripod, is so reduced as to make it unusually light (as is the case in some American patterns), the instrument is practically useless in a breeze, owing to vibration.

Rigidity of construction, steadiness in wind, and capacity for remaining in correct adjustment, more than repay the slight extra weight, which, in any case, should not be allowed to affect the surveyor himself. His time should be occupied with other matters than carrying about an instrument which in any case is comparatively heavy.

A weight of twenty-five pounds for a framed tripod with centering head and a 5-inch instrument cannot be greatly reduced without sacrificing more important qualities to some extent.

### Modifications of the Tacheometer.

Many devices have from time to time been introduced as additions to, or variations of, the ordinary tacheometer. Among them may be mentioned the Wagner-Fennel tacheometer, in which the horizontal distance and the rise or fall in the line of sight are indicated upon horizontal and vertical scales; the "Cleps," in which the telescope is eccentric, the graduation reversed in direction, and the readings are by micrometer; and the "Omnimeter," with micrometers reading into tangent scales. These instruments are very delicate, and their adjustment is difficult; generally speaking, they represent attempts to make the instrument itself solve mechanically the problems of reduction to horizontal dis-

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tance and difference of level. In point of ingenuity they are doubtless admirable, but are not at all suited to ordinary workaday surveying for engineering purposes, and are seldom seen or used.

Two modifications of the tacheometer have, however, recently been introduced which are decidedly practical and advantageous, without in any way detracting from the general usefulness of the instrument as a theodolite. The first of these is found in

### The "Magnum" Tacheometer,

the description of which, given by the makers, is as follows :—

The "Magnum" tacheometer, so named by Mr T. Graham Gribble, M.Inst.C.E., to whose instructions the first instrument was built by Messrs T. Cooke & Sons, Ltd., is, as its name implies, a tacheometer having the greatest possible power consistent with portability.* Its principal features are a telescope of 13-inch focal length, having a clear aperture of 1·8 inch; horizontal limb of 5-inch and vertical limb of 6-inch diameter, both reading by verniers to 20 seconds. Two eyepieces are supplied, the one magnifying twenty and the other forty-two diameters. The lesser power is sufficient for accurate survey up to the distances of 200 metres. The higher power may be used for the same purpose, but it is primarily designed for the measurement of longer distances. A traversing eyepiece is attached, by which means, when using the higher power, the stadia lines are brought well within the field.

Two glass diaphragms are provided, the interchange of which is performed in a simple manner without necessitating any readjustment of the instrument. Both diaphragms

* The details of a survey are usually governed by the main features of the project. The approximate preliminary determination of the latter by means of long-distance measurements will frequently save waste of time in detailed work which is afterwards found unsuitable.

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carry stadia lines, subtending in the ratio of either 1 : 100 or 1 : 200; one of them, however—that for use with the higher power—is subdivided by lines subtending in the ratio of 1 : 4000.

A plummet measuring-tape for finding the height of the transit axis of instrument above ground level is also supplied.

The stadia rod is 3 metres in length, folding in two pieces. It is maintained in a vertical position by a plummet, the cord of which is unwound from a reel at the back of the rod, and, passing through an eyelet, enables the most inexperienced rod-man to hold it true. The rod is graduated to centimetres; with the higher power eyepiece these graduations are visible up to a distance of 400 metres, and the decimetre figures to 1200 metres. The rod is further provided with removable targets, placed at each even metre of its length, at one side. These targets are visible at a distance of over 2000 metres. The approximate determination of very long distances is performed by reading the number of subdivisions on the glass diaphragm subtending one or more metres of the length of rod between the targets. The scope of the survey is thus extended to practically the limit of distance at which the rod-man can be communicated with by signal.

When using the targets, the calculation is as follows :—

Calling the number of subdivisions, with fraction of a subdivision, which subtend any known length of the rod, $m$, then

when one metre on the rod is subtended, the distance $= \frac{4000}{m}$

when two metres, the distance . . . . $= \frac{8000}{m}$

when the full length, the distance . . . . $= \frac{12{,}000}{m}$

*Example.*—One metre on the rod is found to be subtended by 3·9 subdivisions of the glass diaphragm; the distance

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therefore is $\frac{4000}{3{\cdot}9} = 1025$ metres. The fraction of a subdivision is estimated by eye, but a further check is possible.

On a clear day the decimetres on the rod are, at this distance, also visible; a check may therefore be made by observing the length on the rod subtended by an even number of subdivisions. In this case the length on the rod subtended by four full subdivisions would be 1·025 metre, because four subdivisions give a subtense ratio of 1 : 1000.

### The Dempster Tacheometer.

The second modification above referred to is due to Mr H. G. Dempster, Assoc.M.Inst.C.E., and is fully described in an excerpt from *Minutes of Proceedings of the Institution of Civil Engineers*, **187**, which is published in pamphlet form by the Institution.

It consists in extending one of the vertical-circle vernier arms about 35° above and below the horizontal, and engraving thereon, instead of the usual figuring for degrees, etc., a scale of $\sin \times \cos$ in which the divisions have the values 0·01, 0·02, 0·03, etc. A prismatic reader is placed by the side of the telescope and parallel with it, so that a mark upon the horizontal diameter of the vertical arc can be read into the $\sin \times \cos$ scale without moving the head from the eyepiece of the telescope. In taking inclined sights this mark is brought to the nearest division on the scale before reading the webs, much as described on p. 51 in the case of ordinary sexagesimal graduations. The difference of the stadia readings multiplied by the value of the division used on the $\sin \times \cos$ scale, *i.e.* 0·01, 0·02, etc., gives the "rise or fall of the line of sight." Horizontal distances can be worked out by the use of a very small table of "reductions per cent.," involving only one decimal place in the multipliers. By the use of a set of ingenious scales, designed by the inventor, the difference of the stadia readings, their

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mean (which should agree with the axial wire reading, unless the vertical angles are large), the rise or fall of the line of sight, and from it the altitude of the points observed, can all be obtained mechanically. The only disadvantage of the system appears, to the author, to consist in adding *more* weight (*i.e.* that of the extended scale and reader) to the vertical arc end of the telescope axis. The weight of the arc and clamps *alone* is rarely properly counterbalanced, as will be seen if the plate bubbles of an ordinary instrument are accurately centred, with the vertical arc over the A vernier of the horizontal circle, and the telescope axis is then reversed end for end so that the arc lies over the other vernier ( B ). The bubbles will thereupon usually leave the centre of their run by one or more divisions (see p. 123).

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## CHAPTER IX.

### *THE SLIDE-RULE.*

By the use of this little instrument a solution can be obtained mechanically, and with sufficient accuracy for most practical purposes, of any problem which is capable of solution by the use of logarithms ; it is therefore applicable to all calculations in which multiplication and division are involved.

Problems involving addition and subtraction of numbers, or the combined use of logarithms and natural numbers, are not usually performed by its means, as in their case greater speed or accuracy can be attained otherwise, but they are capable of solution by using the " scale of equal parts " on a slide-rule.

It is not possible or necessary in this book to give any detailed exposition of the innumerable applications of the principles involved ; an explanation of those principles themselves, and their special application to tacheometry, must suffice. Readers desirous of going more deeply into the matter are recommended to study the books of instruction issued by makers with the various patterns of slide-rule now upon the market, and other special literature upon the subject.

### General Principles.

The whole system is founded on multiplication and division of numbers by the mechanical addition and subtraction of lengths which graphically represent the logarithms of those numbers.



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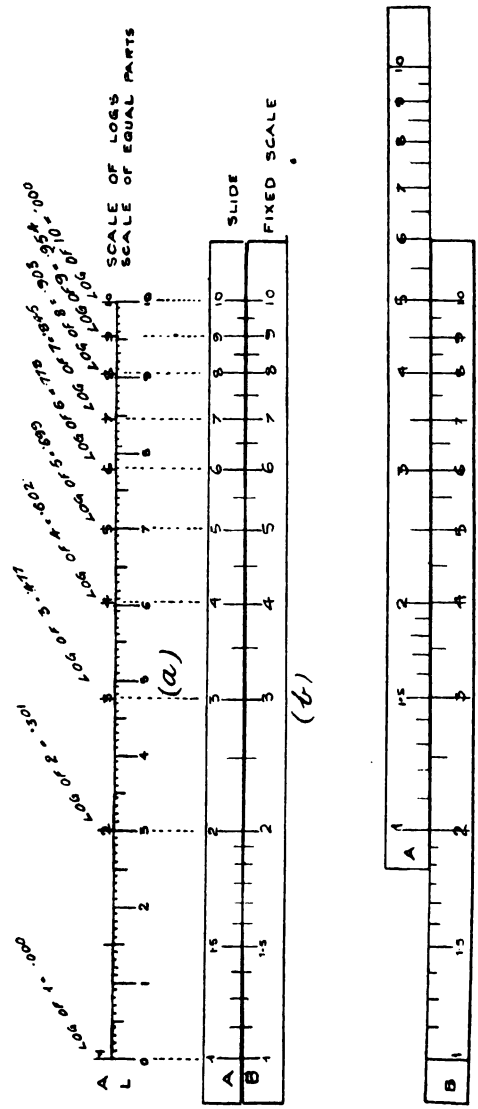


FIG. 28.—Slide-rule Scales

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To illustrate this principle, draw any scale of equal parts, divided to tenths such as scale L, in fig. 28, and set off by it the logarithms of numbers from 1 to 10 to form scale A, placing against each mark on A the *number* whose logarithm it indicates.

## Multiplication and Division.

The sum of the logarithms of numbers is the logarithm of their product, and on the scale A just made it is possible, by stepping off with the dividers, to add together the *spaces* representing the logs of the figures marked, thus arriving at that whole length, composed of these spaces, which will represent the log of their product, and has the number to this log marked against it on the scale. Thus, if the length 1 to 2, representing the log of 2 on scale A, is stepped off to the right of the division marked 3, thus adding the length 1–2 to the length 1–3, there is obtained the length 1–6 representing the log of 6, and marked with that number; similarly, if the length from 1 to 2 on Scale A is taken in the dividers, and stepped off *twice* to the right of the mark 2 on the same scale, the division marked 8, which is the product of $2 \times 2 \times 2$ is reached.

Instead of stepping off with the dividers, it will be more convenient to make a pair of scales, A and B fig. 28 (*b*), both graduated exactly as is scale A in fig. 28 (*a*). The required lengths can then be added together by sliding one scale of the pair along the other. Suppose that Scale A is moved along scale B until its index, 1, is opposite the division marked 2 on B (as in fig. 28 (*c*)). Then against 2, 3, 4, and 5 on A are found 4, 6, 8, 10 respectively on scale B—that is, on scale B and against any number on A is found the product of that number and the figure on B to which the index is set. The figures can be read as 2, 4, 6, 8, etc., or 20, 40, 60, 80, and so on, since the mantissæ, or decimal parts of the logarithms only, are represented by the scales, and not the characteristics,

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which indicate the place of the decimal point. Hence the necessity for the rule given below as to the number of integers in the answer sought, *i.e.* as to the position of the decimal point.

The question also arises: "How is the product of, say, $2 \times 8$ to be obtained, seeing that (fig. 28(*c*)) the number 8 on the sliding scale is beyond the end of the scale on B when the index 1 of A is set to 2 on B?" In this case it is only necessary to use the index (10) at the other end of the sliding scale, moving scale A to the *left* until its last mark 10 is over 8 on B, whereupon there appears 16 on B, against the figure 2 on scale A.

The operation of division is naturally the converse of multiplication.

The rules for multiplication and division with the paper scales just described can be stated as follows:—

*Multiplication.*—"Set 1 or 10 on A to multiplier on B, and against multiplicand on A read product on B."

*Division.*—"Set divisor on A to dividend on B and read quotient on B against 1 or 10 on A." (In division, lengths representing logs are simply subtracted instead of added.)

In the very simple cases of multiplication, etc., considered so far, the number of figures in the answer has been self-evident, but many instances will occur in using decimals where that is not so. It will be best to regard the scale readings as a mere series of figures (since, for example, the figure 18 on the scale can be read as ·18, or as 1·8, 18, 180, 1800, etc., the mantissa of the log being the same in each case), and to fix the place of the decimal point as follows:—

## Position of Decimal Point.

In working by logarithms in the ordinary way, "characteristics" are used to determine the position of the decimal point. In the case of sliding scales, the same principle can

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be adopted by making the characteristic of any number greater by 1 than its log characteristic. This makes the slide-rule characteristic of :—

| | | | | | | |
|---|---|---|---|---|---|---|
| 1000 | to be | 4 | (instead of its log characteristic, which is | | | 3) |
| 100 | ,, | 3 | ( | ,, | ,, | 2) |
| 10 | ,, | 2 | ( | ,, | ,, | 1) |
| 1 | ,, | 1 | ( | ,, | ,, | 0) |
| 0·1 | ,, | ±0 | ( | ,, | ,, | −1) |
| ·01 | ,, | −1 | ( | ,, | ,, | −2) |
| ·001 | ,, | −2 | ( | ,, | ,, | −3) |

Thus, when a number contains *one or more integral figures*, its *slide-rule characteristic* is the same as the number of such integral figures, and is positive ( + ).

When a number is *entirely decimal*, its slide-rule characteristic is the same as the number of ciphers between the decimal point and the first significant figure, and is negative (−). We have the following rules for determining the position of the decimal point (using *slide-rule* characteristics) :—

## Characteristic of Answer.

| If *Index* 1 used (slide projects to right). | If *Index* 10 used (slide projects to left). |
|---|---|
| *Multiplication.* (Characteristic of multiplier plus that of multiplicand) − 1. | (Characteristic of multiplier plus that of multiplicand.) |
| *Division.* (Characteristic of dividend minus that of divisor) + 1. | (Characteristic of dividend minus that of divisor.) |

A few examples are given to illustrate the use of these rules with the paper scales A and B described above.

[TABLE.

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| Calculation. | Side on which Slide projects. | Characteristic of Multiplicand or Dividend. | Characteristic of Multiplier or Divisor. | Characteristic of Answer. | Number of Integers before Decimal Point. | Number of 0s between Decimal Point and First Significant Figure. | Answer. |
|---|---|---|---|---|---|---|---|
| $200 \times 4$ | Right. | (3 + | 1) − 1 | 3 | 3 | ... | 800 |
| $2 \times 40$ | ,, | (1 + | 2) − 1 | 2 | 2 | ... | 80 |
| $2 \times \cdot 4$ | ,, | (1 + | 0) − 1 | 0 | ... | ... | ·8 |
| $\cdot 2 \times \cdot 4$ | ,, | (0 + | 0) − 1 | −1 | ... | 1 | ·08 |
| $\cdot 02 \times \cdot 4$ | ,, | [(−1) + | 0] − 1 | −2 | ... | 2 | ·008 |
| $800 \times 20$ | Left. | 3 + | 2 | 5 | 5 | ... | 16,000 |
| $8 \times 2$ | ,, | 1 + | 1 | 2 | 2 | ... | 16 |
| $8 \times \cdot 2$ | ,, | 1 + | 0 | 1 | 1 | ... | 1·6 |
| $\cdot 8 \times \cdot 2$ | ,, | 0 + | 0 | 0 | ... | ... | ·16 |
| $\cdot 08 \times \cdot 02$ | ,, | (−1) + | (−1) | −2 | ... | 2 | ·0016 |
| $400 \div 200$ | Right. | (3 − | 3) +1 | 1 | 1 | ... | 2 |
| $\cdot 4 \div \cdot 02$ | ,, | [0 − | (−1)] +1 | 2 | 2 | ... | 20 |
| $\cdot 04 \div \cdot 0002$ | ,, | [(−1) − | (−3)] +1 | 3 | 3 | ... | 200 |
| $200 \div 400$ | Left. | 3 − | 3 | 0 | ... | ... | ·5 |
| $\cdot 2 \div \cdot 04$ | ,, | 0 − | (−1) | 1 | 1 | ... | 5 |
| $20 \div \cdot 004$ | ,, | 2 − | (−2) | 4 | 4 | ... | 5000 |

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## Tacheometric Slide-Rules.

Fig. 29 represents the "Universal" slide-rule. It consists of a stock F on which the fixed scales are engraved, a central slide S carrying the movable scales, and a glass cursor R having a fine line ruled on it, which enables close readings to be taken without fatigue to the eyes, and, acting as a marker, facilitates the setting of numbers on the scales. The top scale shown on it, marked L, corresponds to scale L in fig. 28. From it are set off scale A upon the slide and fixed scale B, which, being similar to one another, correspond to the paper scales A and B in fig. 28, and can be used in exactly the same way.

Scale B is repeated just below scale L and is marked C in the illustration, and reading into C is a scale forming two lines on the slide, marked sin $\times$ cos at the left-hand, and a short scale marked $\cos^2$ at the right-hand end of the slide. The sin $\times$ cos scale is plotted from the L scale by setting off successively spaces proportional to the logarithms of (sine $\times$ cosine) $\theta$ for values of $\theta$, advancing by small intervals of arc from 0·35′ to 45°. The $\cos^2$ scale is similarly made for values of $\cos^2\theta$ from 5° to 45°; these two scales are provided for the special purpose of reducing tacheometric observations, and the process of using them to multiply any number by cos . sin $\theta$ or by $\cos^2\theta$ is identical with the process of multiplying two numbers together on the lower scale A and B.

## Cooke's Tacheometer Slide-Rule.

For tacheometrical purposes *only*, a very handy and convenient slide-rule is made by T. Cooke & Son, Ltd.; it has the advantage of being unencumbered with scales and figures intended for carrying out calculations other than those required in tacheometer surveying. It is therefore exceedingly simple in appearance and use, and can be thoroughly recommended. It is made in both 10-inch and

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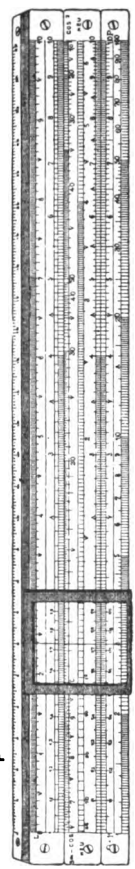


FIG. 29.—"Universal" Slide-Rule.

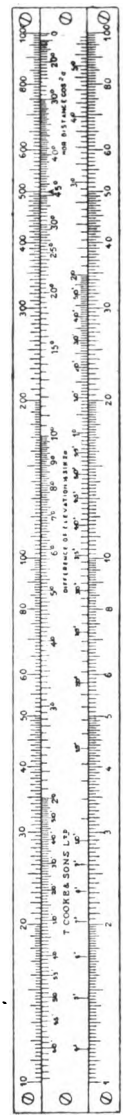


FIG. 30.—Stadia Slide-Rule.

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20-inch lengths, and the dividing being fairly open no cursor is required.

### The Stadia Computer.

Another excellent little device for the horizontal and vertical reduction of readings is a form of circular slide-rule known as the "Stadia Computer." It consists of a graduated revolving card disc and a concentric outer scale or scales; a brass eyelet in the disc enables the user to revolve it with the point of his pencil. When the vertical angle on the one scale is brought to coincide with the observed distance on the other, the desired readings are indicated by the index mark. The computer costs about 4s. 6d., which is much less than any slide-rule; it is very serviceable, but less accurate and much less durable than a wood and celluloid rule.

[TABLE.

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## TABLE OF HORIZONTAL DISTANCES AND VERTICAL HEIGHTS FOR DIFFERENT ANGLES.

| Vertical Angle. | Distance ($\cos^2\theta$). | Height ($\cos\theta \sin\theta$). | Vertical Angle. | Distance ($\cos^2\theta$). | Height ($\cos\theta \sin\theta$). | Vertical Angle. | Distance ($\cos^2\theta$). | Height ($\cos\theta \sin\theta$). |
|---|---|---|---|---|---|---|---|---|
| ° | | | ° | | | ° | | |
| 0·00 | 1·000 00 | ·000 00 | 10·00 | ·969 85 | ·171 01 | 20·00 | ·883 02 | ·321 39 |
| 0·20 | ·999 96 | ·005 82 | 10·20 | ·967 83 | ·176 46 | 20·20 | ·879 26 | ·325 83 |
| 0·40 | ·999 86 | ·011 64 | 10·40 | ·965 74 | ·181 89 | 20·40 | ·875 44 | ·330 22 |
| 1·00 | ·999 69 | ·017 45 | 11·00 | ·963 59 | ·187 30 | 21·00 | ·871 57 | ·334 57 |
| 1·20 | ·999 46 | ·023 26 | 11·20 | ·961 38 | ·192 69 | 21·20 | ·867 65 | ·338 86 |
| 1·40 | ·999 15 | ·029 07 | 11·40 | ·959 11 | ·198 08 | 21·40 | ·863 69 | ·343 13 |
| 2·00 | ·998 78 | ·034 88 | 12·00 | ·956 77 | ·203 37 | 22·00 | ·859 67 | ·347 33 |
| 2·20 | ·998 34 | ·040 68 | 12·20 | ·954 38 | ·208 67 | 22·20 | ·855 60 | ·351 50 |
| 2·40 | ·997 84 | ·046 48 | 12·40 | ·951 92 | ·213 94 | 22·40 | ·851 49 | ·355 60 |
| 3·00 | ·997 26 | ·052 27 | 13·00 | ·949 40 | ·219 18 | 23·00 | ·847 33 | ·359 67 |
| 3·20 | ·996 62 | ·058 04 | 13·20 | ·946 82 | ·224 40 | 23·20 | ·843 12 | ·363 69 |
| 3·40 | ·995 91 | ·063 82 | 13·40 | ·944 17 | ·229 58 | 23·40 | ·838 87 | ·367 66 |
| 4·00 | ·995 13 | ·069 59 | 14·00 | ·941 47 | ·234 73 | 24·00 | ·834 57 | ·371 58 |
| 4·20 | ·994 29 | ·075 34 | 14·20 | ·938 71 | ·239 85 | 24·20 | ·830 22 | ·375 43 |
| 4·40 | ·993 38 | ·081 09 | 14·40 | ·935 89 | ·244 95 | 24·40 | ·825 83 | ·379 26 |
| 5·00 | ·992 40 | ·086 83 | 15·00 | ·933 01 | ·250 00 | 25·00 | ·821 39 | ·383 02 |
| 5·20 | ·991 36 | ·092 55 | 15·20 | ·930 07 | ·255 02 | 25·20 | ·816 92 | ·386 73 |
| 5·40 | ·990 25 | ·098 26 | 15·40 | ·927 08 | ·260 01 | 25·40 | ·812 39 | ·390 39 |
| 6·00 | ·989 07 | ·103 96 | 16·00 | ·924 02 | ·264 96 | 26·00 | ·807 83 | ·394 00 |
| 6·20 | ·987 83 | ·109 64 | 16·20 | ·920 91 | ·269 88 | 26·20 | ·803 23 | ·397 56 |
| 6·40 | ·986 52 | ·115 31 | 16·40 | ·917 74 | ·274 75 | 26·40 | ·798 58 | ·401 06 |
| 7·00 | ·985 15 | ·120 96 | 17·00 | ·914 52 | ·279 59 | 27·00 | ·793 89 | ·404 51 |
| 7·20 | ·983 71 | ·126 60 | 17·20 | ·911 24 | ·284 40 | 27·20 | ·789 17 | ·407 90 |
| 7·40 | ·982 20 | ·132 22 | 17·40 | ·907 90 | ·289 17 | 27·40 | ·784 40 | ·411 24 |
| 8·00 | ·980 63 | ·137 82 | 18·00 | ·904 51 | ·293 90 | 28·00 | ·779 60 | ·414 52 |
| 8·20 | ·978 99 | ·143 40 | 18·20 | ·901 06 | ·298 57 | 28·20 | ·774 75 | ·417 74 |
| 8·40 | ·977 29 | 148 97 | 18·40 | ·897 56 | ·303 22 | 28·40 | ·769 88 | ·420 91 |
| 9·00 | ·975 53 | ·154 50 | 19·00 | ·894 01 | ·307 83 | 29·00 | ·764 96 | ·424 02 |
| 9·20 | ·973 70 | ·160 03 | 19·20 | ·890 40 | ·312 39 | 29·20 | ·760 01 | ·427 08 |
| 9·40 | ·971 81 | ·165 54 | 19·40 | ·886 74 | ·316 92 | 29·40 | ·755 02 | ·430 07 |
| | | | | | | 30·00 | ·750 00 | ·433 03 |

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# FIELD NOTES FOR PLATE I.

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# FIELD NOTES FOR PLATE I.

*Name and Place of Survey*—KINKA RIVER, TRANSVAAL. *Surveyor*—J. SMITH. *Date*—13/4/15.

ON FARM GRANBY. No. 474.

| STATION. FROM | STATION. TO | BEARING. | STADIA WIRES. | OBSD. DIST. | HOR. DIST. | VERT. CIRCLE. | HT. OF INST. | R.L. OF INST. COLL. | LINE OF SIGHT. RISE. + | LINE OF SIGHT. FALL. − | R.L. OF STAFF COLL. | CENTRE WEB. | REDUCED LEVEL. | REMARKS. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ° ′ ″ | | | | | | | | | | | | (*Stadia Webs 1 : 100.*) |
| *B* | .. | .. | .. | .. | .. | .. | *4·65* | *1009·7°* | .. | .. | .. | .. | *1005·05* | *Iron stake and beacon of loose stones.* |
| | *A* | *10 14 30* | *12·58 / 6·83* | *575* | .. | .. | .. | ,, | .. | .. | .. | *9·70* | *1000·00* | *Assumed R.L. for A. Iron stake and beacon of loose stones.* |
| | *C* | *179 47 30* | *7·18 / 3·04* | *414* | .. | .. | .. | ,, | .. | .. | *1009·70* | *5·11* | *1004·59* | |
| | | *348 44 00* | *7·15 / 1·15* | *600* | .. | .. | .. | ,, | .. | .. | *1009·70* | *4·15* | *1005·55* | *At angle in fence. On S. boundary fence of "Fairview."* |
| | | *30 10 00* | *12·60 / 6·40* | *620* | .. | *−30′* | .. | ,, | .. | *5·40* | *1004·30* | *9·50* | *994·80* | *Top of right bank of river.* |
| | | *30 30 00* | *13·27 / 6·97* | *630* | .. | *−1°* | .. | ,, | .. | *10·98* | *998·72* | *10·12* | *988·60* | *Water level at foot of R.B.* |
| | | *335 20 00* | *14·00 / 5·40* | *860* | .. | *+1°* | .. | ,, | *15·00* | .. | *1024·70* | *9·70* | *1015·00* | *On S. boundary fence of "Fairview."* |
| | | *315 00 00* | *7·08 / 2·08* | *500* | .. | *+1°* | .. | ,, | *8·72* | .. | *1018·42* | *4·58* | *1013·84* | |
| | | *348 30 00* | *10·82 / 8·02* | *280* | .. | *+1° 30′* | .. | ,, | *7·32* | .. | *1017·02* | *9·42* | *1007·60* | |
| | | *20 45 00* | *10·06 / 7·14* | *292* | .. | .. | .. | ,, | .. | .. | *1009·70* | *8·60* | *1001·10* | |

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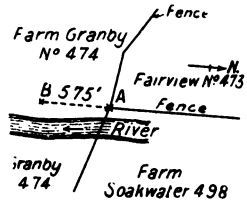
| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 45 36 00 | 8·76 / 5·74 | 302 | .. | −1° 30′ | .. | ,, | .. | 7·90 | 1001·80 | 7·25 | 994·55 | Top of right bank of river. |
| | | 46 35 00 | 11·36 / 8·16 | 320 | .. | −2° 00′ | .. | ,, | .. | 11·15 | 998·55 | 9·76 | 988·79 | W.L. at foot ,, ,, ,, |
| C | .. | .. | .. | .. | .. | .. | 5·26 | 1009·85 | .. | .. | .. | .. | 1004·59 | |
| | B | 359 47 30 | 6·87 / 2·73 | 414 | .. | .. | .. | ,, | .. | .. | 1009·85 | 4·80 | 1005·05 | |
| | D | 181 10 00 | 5·87 / 0·97 | 490 | .. | +1° | .. | ,, | 8·55 | .. | 1018·40 | 3·42 | 1014·98 | |
| | | 270 30 00 | 8·72 / 3·52 | 520 | .. | +1° 30′ | .. | ,, | 13·60 | .. | 1023·45 | 6·12 | 1017·33 | |
| | | 270 30 00 | 7·68 / 5·12 | 256 | .. | +1° 30′ | .. | ,, | 6·70 | .. | 1016·55 | 6·40 | 1010·15 | |
| | | 100 45 00 | 15·90 / 13·80 | 210 | .. | .. | .. | ,, | .. | .. | 1009·85 | 14·85 | 995·00* | * Top of River Bank at W.End of Weir |
| | | 101 20 00 | 14·19 / 11·89 | 230 | .. | −2° | .. | ,, | .. | 8·03 | 1001·82 | 13·04 | 988·78 | W.L. (top of weir is 0·25′ lower). |
| | | 155 42 00 | 12·97 / 9·93 | 304 | .. | −1° | .. | ,, | .. | 5·30 | 1004·55 | 11·45 | 993·10 | Top of river bank. |
| | | 142 15 00 | 16·00 / 12·60 | 340 | .. | −2° 30′ | .. | ,, | .. | 15·34 | 994·51 | 14·32 | 980·19 | W.L. at edge of stream in rocky bed. Rocks run to foot of R.B. |
| | | 148 30 00 | 13·96 / 10·46 | 350 | .. | −2° 30′ | .. | ,, | .. | 15·25 | 994·60 | 12·21 | 982·39 | Foot of R.B. |
| | | 225 00 00 | 10·62 / 4·40 | 620 | .. | +1° 30′ | .. | ,, | 16·22 | .. | 1026·07 | 7·52 | 1018·55 | |
| | | 200 30 00 | 12·42 / 8·32 | 410 | .. | +1° 30′ | .. | ,, | 10·72 | .. | 1020·57 | 10·37 | 1010·20 | |
| | | 190 57 00 | 11·56 / 8·54 | 302 | .. | +1° 00′ | .. | ,, | 5·24 | .. | 1015·09 | 10·05 | 1005·04 | |

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## FIELD NOTES FOR PLATE I—*continued.*

| Station. From | To | Bearing. | Stadia Wires. | Obsd. Dist. | Hor. Dist. | Vert. Circle. | Ht. of Inst. | R.L. of Inst. Coll. | Line of Sight. Rise. + | Line of Sight. Fall. − | R.L. of Staff Coll. | Centre Web. | Reduced Level. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ° ′ ″ | | | | | | | | | | | | |
| *D* | .. | .. | .. | .. | .. | .. | *4·80* | *1019·78* | .. | .. | .. | .. | *1014·98* | |
| | *C* | *1 10 00* | *9·09 / 4·19* | *490* | .. | *−1°* | .. | ,, | .. | *8·55* | *1011·23* | *6·64* | *1004·59* | |
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| *E* | .. | .. | .. | .. | .. | .. | *4·57* | *1024·60* | .. | .. | .. | .. | *1020·03* | |
| | *D* | *359 15 30* | *11·77 / 7·47* | *430* | .. | .. | .. | ,, | .. | .. | *1024·60* | *9·62* | *1014·98* | |
| | *E* | *69 35 00* | *13·66 / 10·73* | *293* | *292* | *−3° 30′* | .. | ,, | .. | *17·80* | *1006·80* | *12·20* | *994·60* | *Top of right bank, on line E–F.* |
| | *F* | *69 35 00* | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | *See station E′ for distance and level of station F.* |
| | | *250 38 00* | *9·86 / 4·98* | *488* | .. | *+1°* | .. | ,, | *8·35* | .. | *1032·95* | *7·42* | *1025·53* | *South fence of "Granby," which is N. fence of "Southfield," No. 475.* |
| | | *181 00 00* | *5·09 / 4·09* | *100* | .. | .. | .. | ,, | .. | .. | *1024·60* | *4·59* | *1020·01* | *South fence of "Granby," at angle in fence.* |
| *E′* | .. | .. | .. | .. | .. | .. | *4·72* | *999·32* | .. | .. | .. | .. | *994·60* | |

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| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | *E* | 249 35 00 | $\frac{8.58}{5.63}$ | 295 | 292 | +5° 30′ | .. | ,, | 27·82 | .. | 1027·14 | 7·11 | 1020·03 | |
| | *F* | 69 30 30 | $\frac{5.84}{0.02}$ | 582 | .. | .. | .. | ,, | .. | .. | 999·32 | 2·62 | 996·70 | |
| | | 20 00 00 | $\frac{14.92}{12.42}$ | 250 | .. | −1° 30′ | .. | ,, | .. | 6·54 | 992·78 | 13·67 | 979·11 | *W.L. at R.B. edge of stream, rocky bed.* |
| | | 2 35 00 | $\frac{13.14}{10.66}$ | 248 | .. | −1° 30′ | .. | ,, | .. | 6·48 | 992·84 | 11·90 | 980·94 | *Foot of right bank. Rocky bed.* |
| | | 135 00 00 | $\frac{15.21}{13.19}$ | 202 | .. | −2° 00′ | .. | ,, | .. | 7·12 | 992·20 | 14·20 | 978·00 | *Water level, edge of stream near south fence of "Granby."* |
| | | 180 10 00 | $\frac{7.01}{5.61}$ | 140 | .. | .. | .. | ,, | .. | .. | 999·32 | 6·31 | 993·01 | *Top of right bank.* |
| *F* | .. | .. | .. | .. | .. | .. | 4·70 | 1001·40 | .. | .. | .. | .. | 996·70 | |
| | *E′* | 249 35 00 | $\frac{9.70}{3.90}$ | 580 | .. | .. | .. | ,, | .. | .. | 1001·40 | 6·80 | 994·60 | |
| | *G* | 350 30 30 | $\frac{10.51}{4.49}$ | 602 | .. | +1° | .. | ,, | 10·50 | .. | 1011·90 | 7·50 | 1004·40 | |
| | | 161 20 00 | $\frac{6.67}{3.63}$ | 304 | .. | +1° | .. | ,, | 5·30 | .. | 1006·70 | 5·15 | 1001·55 | *S. fence of "Granby" or N. fence of "Southfield."* |
| | | 200 00 00 | $\frac{9.83}{6.95}$ | 288 | .. | .. | .. | ,, | .. | .. | 1001·40 | 8·39 | 993·01 | *Top of vertical left bank.* |
| | | 94 00 00 | .. | .. | .. | +2° | *General slope angle of surface in this direction.* | | | | | | | |
| | | 278 40 00 | $\frac{9.30}{7.30}$ | 200 | .. | .. | .. | .. | .. | .. | 1001·40 | 8·30 | 993·10 | *Top of vertical L. bank, which ends here, and becomes a rocky slope upstream of this point.* |
| | | 330 25 00 | $\frac{9.96}{6.34}$ | 362 | .. | .. | .. | .. | .. | .. | 1001·40 | 8·15 | 993·25 | |
| | | 325 00 00 | $\frac{13.30}{9.56}$ | 374 | .. | .. | .. | .. | .. | .. | 1001·40 | 11·43 | 989·97 | |
| | | 325 00 00 | $\frac{15.33}{10.59}$ | 474 | .. | −1° | .. | .. | .. | 8·27 | 993·13 | 12·96 | 980·17 | *W.L. at L.B. edge of stream on rocky bed.* |

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## FIELD NOTES FOR PLATE I—*continued.*

| Station. From | Station. To | Bearing. | Stadia Wires. | Obsd. Dist. | Hor. Dist. | Vert. Circle. | Ht. of Inst. | R.L. of Inst. Coll. | Line of Sight. Rise. + | Line of Sight. Fall. − | R.L. of Staff Coll. | Centre Web. | Reduced Level. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| *F* | *G* | ° ′ ″ | | | | | | | | | | | | |
| | | *240 30 00* | *9·00 / 7·20* | *180* | .. | .. | .. | .. | .. | .. | *1001·40* | *8·10* | *993·30* | *Top of bank.* |
| | | *300 10 00* | *14·82 / 11·00* | *382* | .. | *−40′* | .. | .. | .. | *4·44* | *996·96* | *13·21* | *983·75* | *On rocks at edge of stream, not W.L.* |
| | | *46 00 00* | *3·42 / 0·38* | *304* | .. | *+2°* | .. | | *10·60* | .. | *1012·00* | *1·90* | *1010·10* | |
| | | *340 25 00* | *6·34 / 1·70* | *464* | .. | .. | .. | .. | .. | .. | *1001·40* | *5·18* | *996·22* | *Top of rough rocky bank.* |
| *G* | .. | .. | .. | .. | .. | .. | *4·71* | *1009·11* | .. | .. | .. | .. | *1004·40* | |
| | *F* | *170 30 30* | *15·43 / 9·40* | *603* | .. | .. | .. | ,, | .. | .. | *1009·11* | *12·41* | *996·70* | |
| | *H* | *13 45 30* | *7·16 / 0·94* | *622* | .. | .. | .. | ,, | .. | .. | *1009·11* | *4·05* | *1005·06* | |
| | *B* | *301 22 00* | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | |
| | *A* | *328 32 30* | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | |
| | *D* | *237 6 30* | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | |
| | *C* | *271 47 30* | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | |
| | | *90 00 00* | | .. | .. | *+1° 30′* | *General slope angle of ground surface.* | | | | | | | |
| | | *280 35 00* | *14·23 / 12·11* | *212* | .. | *−2° 00′* | .. | *1009·11* | .. | *7·40* | *1001·71* | *13·17* | *988·54* | *East end of Weir Crest, built into rocky bank.* |
| | | *283 00 00* | *15·05 / 12·13* | *292* | .. | .. | .. | .. | .. | .. | *1009·11* | *14·09* | *995·02* | *On rocky bank at top. Shelving bank of rock ends about 30′ upstream of this point, and bank is vertical beyond.* |

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| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 250 00 00 | 12·03 / 8·49 | 354 | .. | −2° 30′ | .. | ,, | .. | 15·43 | 993·68 | 10·26 | 983·42 | West side of Rocky Island about 150×50′   |
| | | 272 00 00 | 14·52 / 11·48 | 304 | .. | −2° 00′ | .. | ,, | .. | 10·60 | 998·51 | 13·00 | 985·51 | *Foot of Weir, downstream side.* |
| | | 340 15 00 | 15·02 / 10·96 | 406 | .. | .. | .. | ,, | .. | .. | 1009·11 | 12·99 | 996·12 | *Edge of bank.* |
| | | 320 36 00 | 15·01 / 12·38 | 263 | .. | .. | .. | ,, | .. | .. | 1009·11 | 13·69 | 995·42 | ,, ,, |
| | | 13 45 30 | 5·92 / 2·70 | 322 | .. | .. | .. | ,, | .. | .. | 1009·11 | 4·31 | 1004·80 | |
| | | 39 20 00 | 8·68 / 5·56 | 312 | .. | +1° 30′ | .. | ,, | 8·16 | .. | 1017·27 | 7·12 | 1010·15 | |
| | | 39 20 00 | 9·68 / 4·58 | 510 | .. | +1° 30′ | .. | ,, | 13·35 | .. | 1022·46 | 7·13 | 1015·33 | |
| H | .. | .. | .. | .. | .. | .. | 4·80 | 1009·86 | .. | .. | .. | .. | 1005·06 | |
| | G | 193 45 30 | 8·57 / 2·33 | 624* | .. | .. | .. | ,, | .. | .. | 1009·86 | 5·46 | 1004·40 | *Distance should be 622. Stadia reading 2·33 should be to 2·35. Cf. G–H above. |
| | A | 297 37 00 | 14·13 / 5·55 | 858 | .. | .. | .. | ,, | .. | .. | 1009·86 | 9·84 | 1000·02 | |
| | | 13 45 30 | 5·12 / 4·12 | 100 | .. | .. | .. | ,, | .. | .. | 1009·86 | 4·62 | 1005·24 | *South fence of Soak water No. 498.* |
| | | 267 17 00 | 13·87 / 10·85 | 302 | .. | .. | .. | .. | .. | .. | 1009·86 | 12·36 | 997·50 | } *Top L.B of river.* |
| | | 297 37 00 | 13·66 / 10·84 | 282 | .. | .. | .. | ,, | .. | .. | 1009·86 | 12·25 | 997·61 | |
| | | 90 15 00 | | 306 | .. | +2° 0′ | .. | ,, | 10·68 | .. | 1020·54 | 5·02 | 1015·52 | *General slope of country in this direction is +2°.* |
| A | .. | .. | .. | .. | .. | .. | 4·70 | 1004·72 | .. | .. | .. | .. | 1000·02 | |
| | H | 117 37 00 | 11·43 / 2·85 | 858 | .. | +30′ | .. | ,, | 7·47 | .. | 1012·19 | 7·14 | 1005·05 | } *To check bearings, etc.* |
| | B | 190 14 30 | 7·57 / 1·82 | 575 | .. | +30′ | .. | ,, | 5·01 | .. | 1009·73 | 4·69 | 1005·04 | |

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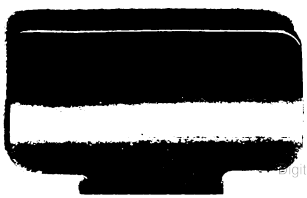

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